Seismic Performance of Damaged Bridge Columns

S. J. Elkin¹⁾

D. E. Lehman¹⁾ J. P. Moehle²⁾

1) Graduate Student Researcher, Dept. of Civil and Environmental Engineering, University of California, Berkeley.

A. M. Nacamuli¹⁾

2) Professor, Dept. of Civil and Environmental Engineering, University of California, Berkeley, Earthquake Engineering Research Center, 1301 South 46th Street, Richmond, CA 94804, U.S.A.

ABSTRACT

It is vital that important highway structures are functional following an earthquake. During strong earthquake shaking, columns supporting bridge structures may be expected to form inelastic flexural hinges at critical locations. Restoration of highway structures to serviceable conditions may require replacement or repair of damaged regions. Significant research has focused on repair and retrofit of columns meeting design standards prior to the 1970's. However, limited experimental studies have considered methods to repair damaged bridge columns designed to current standards.

An experimental study to evaluate repair techniques for damaged bridge columns was undertaken. The repair program consisted of four repair test specimens: three damaged to a severe level and one damaged to a moderate level. Repair schemes for the severely damaged columns used headed reinforcement and mechanical couplers. The moderately damaged column was repaired by cover replacement and epoxy injection. The success of each scheme is evaluated by comparing behavior of the repaired column with that of the original column.

INTRODUCTION

The assumption made in current conventional seismic design practice is that bridge columns will form ductile flexural hinges at predetermined locations and sustain numerous inelastic reversed loading cycles during design level ground shaking. As a result, reinforced concrete bridge columns will experience some level of damage after a seismic event. This research project addresses the ability of specific repair techniques to restore a reinforced concrete bridge column, damaged by seismic loading, to its pre-damage condition. The repair methods and test results are specifically applicable to damaged columns that were designed using current ductile detailing practice.

The testing program involved repairing and testing four previously damaged columns. The emphasis was on developing constructable repair schemes that achieved the performance levels of the virgin columns.

OVERVIEW OF EXPERIMENTAL PROGRAM

An experimental study was conducted to evaluate current design procedures for reinforced concrete bridge columns. Three one-third-scale specimens, with varying quantities of longitudinal reinforcement ratios, were constructed and tested. The areas of longitudinal reinforcement considered were 0.0075, 0.015 and 0.030 times the gross area. The latter column was detailed with bundled All other details, including hars geometry, transverse reinforcement size and spacing, longitudinal reinforcement size, loading, and instrumentation, were constant for the three columns [1].

The repair project involved repairing these previously damaged columns. Each of these three columns had been tested to failure (strength reduction exceeding 20%); their final damage condition was considered severe. А fourth column, which was identical to one of the previous columns, was constructed and tested to a moderate damage level. The additional column had a longitudinal reinforcement ratio of 1.50%. The reinforcement ratio for the additional test was considered representative of an "average" bridge column.

Original Column Design

The original columns were designed using current Caltrans design procedures [2], as well as proposed revisions to that document [3]. Details of these original test specimens are shown in Fig. 1.

The columns were two feet in diameter and eight feet in height. Table 1 summarizes the column geometry and reinforcement for the test specimens. Design analysis of the prototype columns can be found elsewhere [4].



Fig. 1 Virgin column geometry and reinforcement

Column Designation	Column Geometry			Longitudinal			Transverse		
	Diameter	Length	Block	$ ho_1$	d_b	#	$ ho_{ m s}$	d_s	s
407	2'-0"	8'-0"	$8'\times4'\times2'$	0.75%	5/8″	11	0.70%	2/8″	1-1/4″
415	2'-0"	8'-0"	$8'\times4'\times2'$	1.50%	5/8″	22	0.70%	2/8″	1-1/4″
430	2'-0"	8'-0"	$8'\times4'\times2'$	3.00%	5/8″	44	0.70%	2/8″	1-1/4″

Table 1 Geometry and reinforcement of virgin specimens

Damage States

Current design recommendations describe two performance levels for bridge structures [3]. These performance levels are described qualitatively below:

ATC-32: C.3.21.2.3.b

- Repairable Damage: Inelastic response may occur, resulting in concrete cracking, reinforcement yield and minor spalling of cover concrete. The extent of damage should be sufficiently limited that the structure can be restored essentially to its preearthquake condition without replacement of reinforcement or replacement of structural members.
- Significant Damage: Although there is minimum risk of collapse, permanent offsets may occur and damage consisting of cracking, reinforcement yielding, and major spalling of concrete may require closure to repair. Partial or complete replacement may be required in some cases.

The damaged conditions of the test specimens for this project are representative of the damage levels described in the ATC-32 document. Repairable and significant damage levels are referred to as moderate and severe damage, respectively (Table 2).

Test Matrix

The repair and test program involved four test specimens. Three of the columns were indicative of severe damage, the fourth of moderate damage. The columns comprising the severe damage exhibited concrete cracking, series spalled concrete, crushed concrete, bar buckling, and fracture of longitudinal and spiral reinforcement. The moderately damaged column exhibited concrete cracking, spalled concrete, and yielding of the reinforcing steel. Additional details of the observed damage levels are summarized in Table 2.

Column designation	Concrete damage						
	Spalled height (in)	Core crush depth (in)	Yielding of long. bars	No. of buckled long. bars	No. of fractured long. bars	No. of fractured spirals	Damage level
407	14	2	All bars	7	5	8	Severe
415M *	15	0	Extreme bars	0	0	0	Moderate
415S *	18	7	All bars	22	9	4	Severe
430	15	8	All bars	44	0	8	Severe

Table 2 Summary of column damage

* With the exception of the final damage level, Columns 415M and 415S were identical to one another.



Fig. 2 Experimental configuration

The four columns were each repaired using a different technique. The three repair schemes for the severely damaged columns include: replacing the damaged region, jacketing the existing section to restrict inelastic action outside of the jacketed region, and jacketing the existing section to restrict inelastic action within the jacketed region. The moderately damaged column was repaired using standard repair techniques of cover replacement and epoxy injection.

Experimental Testing Program

The columns were tested upright and modeled a fixed base cantilever (Fig. 2). The columns were subjected to lateral and axial load. The applied vertical load was equal to ten percent of the grosssection capacity (0.10 $A_g f_c$). Lateral load was applied in a reversed cyclic manner under displacement control. All of the specimens, both virgin and repaired, were subjected to the same axial load and displacement history.

EXPERIMENTAL RESULTS

Column 407R

The column was repaired by replacing

damaged regions of the column and joint with new concrete and reinforcement with mechanical couplers.

Description of Damage State

Column damage included cover spalling to approximately half the column diameter. Maximum crushing of column core and anchor block was limited to 2.5-inch depth. Of the 11 column longitudinal bars, five fractured following bar buckling and two buckled without fracture. The spiral reinforcement in the spalled region fractured at several locations.

Design Methodology

The majority of the longitudinal reinforcement had fractured and/or buckled. To recover the original strength and ductility, the damaged steel and concrete were replaced. To restore the original column stiffness, the "inelastic range" of the column (including longitudinal reinforcement, spiral, core and cover concrete) was removed and replaced. The regions of the column and joint to be removed were computed using Eqs. (1) and (2).

The computed "inelastic length" is a function of the flexural and tension-shift demands. To define this length, the analytical yield moment, M_y , and ultimate moment, M_u , are determined. The resulting inelastic length, L_y , is proportional to the column length. This length is assumed to increase due to tensionshift effects by a length equal to the column diameter divided by the square root of three as shown in Eq. (1).

$$L_y = \frac{M_u - M_y}{M_u} \times L + \frac{D}{\sqrt{3}} \tag{1}$$

Inelastic action in the joint region includes concrete pullout and crushing as well as yielding of the anchored longitudinal reinforcement. The damaged region in the joint is estimated assuming an equivalent uniform bond strength and over-strength factor for the longitudinal reinforcing steel of 1.25. The length to remove is determined by Eq. (2).

$$l_{yjoint} = \frac{0.25 f_y d_b}{4 \times (6 \sqrt{f_c'})}$$
(2)

Repair Geometry and Reinforcement

Details of the repair scheme are shown in Fig. 3. A 3-foot section of the column was removed. To achieve this, the column was mechanically severed at the interface and 3 feet above the anchor block and removed. Repair of the joint included removing the loose concrete in the joint region and 6 inches of the longitudinal bars anchored into the joint, approximately equivalent to ten bar diameters.



Fig. 3 Column 407R repair design

Following demolition, the remaining portion of the existing column longitudinal reinforcement was connected to the longitudinal reinforcement in the anchor block using mechanical couplers. The repaired portion of the column was tied with new spiral reinforcement, having 3/8-inch diameter spiral at 2 1/4-inch pitch; thus, the resulting reinforcement ratio is equivalent to the original column. Accessibility to the column core during casting required the column be recast to a diameter 4 inches larger than the original column resulting in a cover dimension of $2 \ 3/4$ -inches.

Experimental Observations and Force-Displacement Response

The force-displacement response for the original and repaired columns is

shown in Fig. 4. The response of the repaired specimen was similar to the original with a slight increase in strength and stiffness due to the increased concrete cover. Strength decrease resulted from cover spalling on the repaired specimen. The post-spalling strength tended towards the strength of the original specimen, as expected. The ultimate and yield displacements of the repaired column were greater than the corresponding quantities in the original The resulting displacement column. ductilities were comparable.



Fig. 4 Column 407R force-displacement response comparison of virgin *vs.* repaired response

Column 415SR

The repair technique used for this column involved moving the hinge location above the previously damaged regions.

Description of Damage State

The displacement capacity of the original column was 7 inches, resulting in a displacement ductility of approximately 7. However, following completion of testing, the actuator accidentally moved to 10 inches resulting in additional damaged.

Concrete damage included spalled

column cover up to 18 inches, crushed core concrete to a depth of 7 inches, anchor block damage to a depth of 2 inches below the interface, and significant cracking throughout the column height. All of the 22 column longitudinal bars had buckled and nine had fractured. Steel damage was more pronounced on one side of the column; seven longitudinal bars and three levels of spirals had fractured as a result of the additional actuator movement.

Design Methodology

The repair scheme restricted inelastic action outside of the previously damaged region by moving the hinge up the column height. Movement of the column hinge caused a reduction in the effective column height. Consequently, the flexural strength was necessary to ensure the plastic shear demand would not increase. Therefore, the moment capacity at the new hinge region was decreased and the previously damaged region was strengthened with a concrete jacket.

The jacket height is chosen to minimize the reduction of the effective column height, while effectively replacing the damaged region. Substantial concrete damage had occurred within the lower 18 inches of column height, and steel damage was significant within the lower 8 inches of column height. Therefore, a jacket height of 22 inches was chosen to effectively repair the previously damaged section.

To regain the original strength, the reduced moment demand for the new inelastic region was computed by multiplying the original shear demand, V_n , and the new effective column height, L_e . To achieve the decreased moment capacity six of the existing longitudinal bars were severed four inches below the

top of the jacket. To ensure proper continuity of the remaining steel, the existing bars that had fractured were repaired with welded lap splice connections.

The moment demand at the base, M_b , was determined by factoring up the moment capacity at the new hinge, M_u , and assuming a linear moment diagram. To ensure the jacketed region remained essentially elastic, the yield strength of the jacket, M_{yj} , was designed to be greater than the resulting base moment, M_b . Longitudinal reinforcing bars with heads on each end were used within the concrete jacket (Fig. 5). Headed bars require a shorter development length than traditional reinforcement, allowing a short jacket height and minimal embedment into the anchor block.



Fig. 5 Column 415SR repair design

Repair Geometry and Reinforcement

The repair scheme is shown in Fig. 5. The damaged concrete was removed from the lower 22 inches of column height and from the anchor block surrounding the column perimeter. Holes were drilled into the anchor block at the locations of the new headed bars. The damaged spiral was removed from the lower 10 inches of column height. Six of the existing longitudinal bars were cut four inches below the top of the new concrete jacket. The remaining fractured bars were repaired with welded lap splices.

Following demolition, ten headed longitudinal bars were grouted into the anchor block with a high strength grout (Chem-75). No. 3 ($^{3}/_{8}$ -inch diameter) spiral reinforcement was placed in the jacketed region at $1 \frac{1}{2}$ -inch pitch, with 2 levels of spiral at the top to prevent kick-out of the top head. The concrete jacket was cast, and the existing cracks in the hinging region were injected with epoxy.

Experimental Observations and Force-Displacement Response

As designed, the inelastic action was restricted above the height of the concrete jacket. The force-displacement response for the original and repaired columns is shown in Fig. 6. The decreased moment capacity at the new hinge resulted in the original column strength with no increase in the shear demand. The decrease in the effective height, resulted in an increase in the column stiffness and a

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decrease in both the displacement capacity and the displacement ductility for the repaired column.



Fig. 6 Column 415SR force-displacement response comparison of virgin *vs.* repaired response

Column 430R

The repair scheme was to place a reinforced concrete jacket around the severely damaged portion of the existing column and limit inelastic action to the jacketed section.

Description of Damage State

Observations of column concrete damage following testing include cover spalling to a height exceeding half the column diameter (13 to 15 inches), and crushing of column core to 8 inches. Damage to the anchor block included concrete spalling at the column face approximately 2 feet long and 4 inches wide with maximum depth of 1 inch. Of the 44 longitudinal reinforcing bars, all buckled and none fractured. The buckled length ranged from 3 to 6 tie spacings (4 to 8 inches).

Design Methodology

The repair scheme was designed to retain plastic hinge formation at the base of the column. The jacket dimensions and reinforcement were selected to ensure yielding occurred within the jacketed region. To eliminate the uncertainty associated with strength of the previously damaged section, the existing longitudinal bars were severed prior to placement of the column jacket. The jacketed portion of the existing column was roughened to ensure adequate shear transfer.

The jacket was longitudinally reinforced with 16 No. 6 bars and 3/8-inch diameter spiral spaced at a 1-inch pitch. The section capacity was estimated to provide an ultimate moment capacity of approximately 857 kip-ft, just slightly greater than the original capacity of 852 kip-ft.

The length of the jacketed section, L_j , required to limit the demand in the existing cross-section (at the top of the jacket) to the yield moment is given by Eq. (4). M_{uj} is the ultimate capacity of the jacketed section, M_{ye} is the flexural strength of the existing section corresponding to first yield of the longitudinal reinforcement, and L is the length of the cantilever column.

$$L_j = \frac{M_{uj} - M_{ye}}{M_{uj}} \times L \tag{4}$$

In addition, the length of the jacketed section must be sufficient to ensure adequate bond transfer from the new longitudinal bars to the existing reinforcement. Using a bi-uniform bond strength of $12\sqrt{f'_c}$ within the elastic range and $6\sqrt{f'_c}$ in the inelastic range of the new headed bar, Eq. (5) gives the required length.

$$L_j = \frac{d_b f_y}{30 \left(\sqrt{f_c'}\right)} \tag{5}$$

 f'_c and f_y are the strengths of the respective strengths of the concrete and

steel in psi and d_b is the diameter of the new longitudinal bar.

Within the joint, the anchorage depth of the headed bar must be determined. Research into the behavior of anchored, headed reinforcement has been limited to monotonic and cyclic tensile loading [5] which is not directly adaptable to seismic applications. Therefore, to ensure adequate anchorage, the expression for anchorage depth, l_a , developed by De Vries and Jirsa was magnified by a factor of 1.33, as given in Eq. (6) (d_b , diameter of the bar in mm).

$$l_d = 1.33 \times (1''/25.4 \text{mm}) \times \left[\frac{28 \times \pi \times d_b^2 f_u}{1000 \sqrt{f_c'}}\right]^{2/3}$$
(6)

Repair Geometry and Reinforcement

Figure 7 shows the final jacket dimensions. Using the above expressions, the final jacket dimensions were determined to be 32 inches in diameter and 3 feet in height (the dimensions satisfy the three expressions with a factor-of-safety of 1.33). The headed bars were embedded 12 inches into the joint.



Fig. 7 Column 430R repair design

Experimental Observations and Force-Displacement Response

Figure 8 presents the force-displacement response of the original and repaired specimens. The repair scheme achieved the performance objectives. There was a slight increase in the initial stiffness of the repaired specimen and the ultimate moment capacity was slightly less than the original column. The low longitudinal reinforcement ratio of the repaired specimen resulted in earlier exhaustion of the tensile strain capacity of the steel, limiting the displacement capacity of the repaired specimen to 5 inches (compared with 7 inches for the original column). However, the displacement ductility capacity of the two specimens was comparable.

Column 415MR

Description of Damage State

Based on experimental results from Column 415, tested to a severe damage state, this column was cycled to the 3inch displacement level using the same displacement history. This was representative of a displacement ductility of approximately 3. The moderate damage

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Fig. 8 Column 430R force-displacement response comparison of virgin vs. repaired response

condition included: spalled cover up to 15 inches on the extreme column faces, cracking throughout the column height, and yielding of the extreme longitudinal reinforcement.

Design Methodology

The repair technique used for this damage condition involved epoxy injection of the cracks and patching of the spalled region. Although previous research has investigated this type of repair technique, it has not been tested on ductile hinge regions of reinforced concrete bridge columns. These methods are used in practice; therefore, it is imperative that their performance is evaluated for use on damaged hinge zones.

Repair Procedure

Damaged concrete was removed from the lower 18 inches of column height. Surfaces were roughened and blown clean along crack lengths and spalled regions. Ports were located along cracks at approximately a six-inch spacing. Sealants were applied to the cracks and the region of spalling. Cracks were injected with a low-viscosity epoxy resin,

Concresive 1360. The maximum crack width that could be injected was 0.003 inches; therefore, the injection could only be performed in the lower two to three feet of column height, the hinging region of the column. Once the epoxy injection process was complete, the spalled region of the column was repaired with a patching material, concrete Thorite. This material was mixed to a low slump repaired with a concrete patching material, Thorite. This material was mixed to a low slump consistency and applied by hand in layers, scoring the surface between each application of a layer to ensure adequate bond. The properties of this material are comparable to concrete properties, without the need for formwork or a long curing time.

Experimental Observations and Force-Displacement Response

The force-displacement response is presented in Fig. 9. The same ultimate strength and ultimate displacement were achieved. However, the stiffness of the required specimen was significantly less than the stiffness of the original column. The difference in column stiffness may be due to a number of factors including: the





existence of cracks above the hinge region that could not be injected, an inability to penetrate epoxy into the core concrete of the hinge region, crushed concrete within the core of the hinge region, previously yielded reinforcing steel, or degradation of bond into the anchor block.

SUMMARY AND CONCLUSIONS

The experimental force-displacement envelopes of the original and repaired columns are shown in Fig. 10. А comparison of the three severely damaged columns shows the repaired columns all had the same or greater stiffness than the original specimens. The repaired strength was within 10% of the original strength for all columns. The deformability of the repaired specimens, with respect to the original columns varied. In the case of 415SR and 430R, the deformation capacity was less. The deformation capacity of column 407R exceeded that of the original specimen.



Fig. 10 Force-displacement envelopes

Comparing the response of 415MR, the moderately damaged column, and 415SR, the severely damaged column with that of the original column provides interesting results. The initial stiffness of the moderately damaged specimen was significantly less than the original. However, the strength and deformability of the repaired specimen was equal to the original. The response of 415SR is markedly different than 415MR. The initial stiffness and strength of the column closely matched the original behavior; however, the displacement capacity was less.

Overall, the repair schemes met their performance objectives. The experimental results prove the ability to repair both severely and moderately damaged ductile reinforced concrete bridge columns.

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