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Hybrid Jacketing for Rapid Repair of Seismically Damaged Reinforced Concrete Columns

Mostafa Fakharifar, Genda Chen, Mahdi Arezoumandi, and Mohamed ElGawady

This study proposes hybrid jacketing for rapid repair of seismically damaged concrete columns for bridge safety. The hybrid jacketing for a reinforced concrete (RC) column is composed of a thin cold-formed steel sheet wrapped around the column and its outside prestressing strands. Although the prestressing strands can prevent buckling of the confining steel sheet, the steel sheet can in turn prevent the prestressing strands from cutting into the concrete. The hybrid jacketing concept was validated with testing of a large-scale RC column with lap splice deficiency typical of pre-1970 bridge constructions in the Central United States. Results from the original and repaired columns were compared for hysteresis loops, strength, stiffness, ductility, and energy dissipation. The hybrid jacketing proved to be effective in restoring structural behavior of the damaged column to prevent bridge collapse. Such a cost-effective solution can be implemented at bridge sites in hours. Design equations to establish the lateral force–displacement relationship of the tested column to design the hybrid jacket are derived in detail.

Bridges are critical links in a surface transportation network. During an earthquake event, they are required to withstand strong ground motions so that emergency personnel and vehicles can be dispatched into the struck area for postearthquake evacuation and response. Lessons learned from past earthquakes testify to the importance of bridge safety in the overall resilience of the highway transportation network. For example, the 1994 Northridge Earthquake caused the collapse of seven highway bridges in Los Angeles, California, and severely damaged many other bridges, resulting in significant disruption on the regional highway transportation network (*1*).

Bridges in seismically active areas are vulnerable to a series of main-shock–aftershock ground motions. The 2011 Tohuku Earthquake with a moment magnitude (M_w) of 9 was succeeded by hundreds of aftershocks, including at least 30 aftershocks greater than M_w 6 (*2*). Because of the frequent occurrence of aftershocks, damaged bridges must be repaired in a short time with innovative techniques.

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Many of the 12,000 bridges in the inventory of the state of California were constructed before the 1971 San Fernando Earthquake with common seismic deficiencies, such as insufficient transverse reinforcement and lap splice reinforcement at column bases (*3*). To improve their safety, reinforced concrete (RC) bridge columns have been retrofitted with various jacketing techniques and materials (*4–11*). However, very few studies addressed the rapid repair of severely damaged RC columns (*12–14*). The past studies for column retrofitting and repair have demonstrated the inadequacy of columns with a lap splice length of 20 d_b (d_b is the reinforcement diameter) and the advantages of external jacketing to prevent potential lap splice failures (*4, 10*).

EXISTING SEISMIC RETROFIT AND REPAIR TECHNIQUES

RC columns can be confined both actively and passively. With active confinement, the confining pressure is applied to concrete columns before the progression of concrete damage (*3, 4, 10*). With passive confinement, concrete is subjected to damage (lateral dilation) before confinement is in effect through the buildup of hoop stress (*4*). Figure 1, *a* and *b,* illustrates the schematics of cross sections of passively and actively confined RC columns, respectively. In general, active confinement can improve the strength and ductility of concrete more significantly than passive confinement (*4, 5*). Active confinement is preferred for inadequate lap-spliced columns because concrete dilation is not required to activate the jacketing pressure, as concrete lateral dilation leads to bond deterioration between lapped bars (*10*). Priestley et al. presented different seismic rehabilitation techniques of RC bridge columns using steel, concrete, fiber-reinforced polymer (FRP), and prestressing strands (*4*). Different jackets, based on the confining pressure provided, are categorized and presented in Figure 1*c.* Past studies proved the efficacy of different confining repair jackets (*4–14*). However, in addition to the high material cost and concerns about the long-term performance (prone to moisture), FRP materials were shown to suddenly rupture because of their linear elastic properties (*4, 12*). Although thick steel jackets were available and economical with ductile behavior, material handling and high field installation costs were their major drawbacks (*3, 4*). Lin et al. successfully implemented a seismic retrofitting system with active confinement using prestressing strands (*5*). However, tests indicated that the columns retrofitted with prestressing strands experienced strength deterioration under cyclic loading caused by high losses of the prestressing force in confining strands. The concrete cover spall

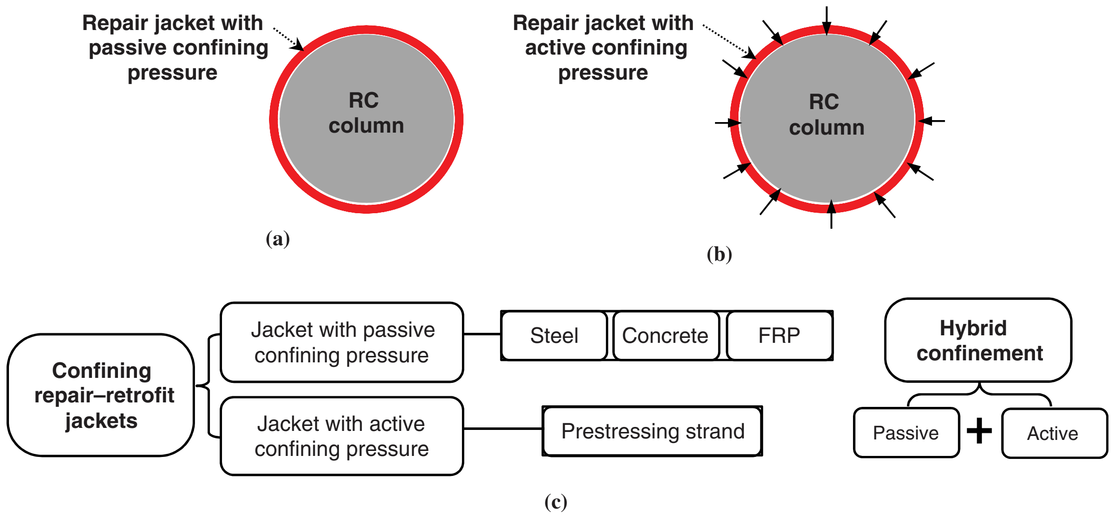


FIGURE 1 Comparison of existing retrofit and repair methods: (*a*) passive confinement, (*b*) active confinement, and (*c*) confining pressure from various repair jackets (FRP = fiber-reinforced polymer).

during cyclic reversals and the penetration of prestressed strands into the concrete caused the loss of confining force.

The above-mentioned studies were focused on conventional thick steel or thin composite jacketing for the seismic retrofitting and repair of RC columns. This study aimed at developing a novel, hybrid passive–active jacketing technology with an inside lightweight steel sheet and outside prestressing strands to incorporate the active confining pressure along with the economical, ductile steel jacket, without requiring heavy equipment for field applications. The hybrid jacket requires no epoxy curing time, is less expensive than FRP wraps, and is significantly lighter than conventional steel or concrete jackets with less labor-intensive installations. The rapid repair technique is validated by designing and fabricating a large-scale RC bridge column, testing the column with substantial damage, repairing it with the proposed hybrid jacketing, and comparing the performances of the original and repaired columns. The hybrid jacket incorporates the additional advantages of both active and passive confinement into one repair jacket.

EXPERIMENTAL PROGRAM

Original Column Design

One half-scale circular RC column was constructed and tested to failure in the High-Bay Structures Laboratory at the Missouri University of Science and Technology. The column represents typical pre-1970s bridge piers with column longitudinal reinforcement lap spliced at column-footing joints. The lap splice length was equal to 20 d_b or 20 in. (508 mm) for No. 8 deformed rebar. As shown in Figure 2, the total height of the column was 167 in. (4,242 mm) with an effective height of 132 in. (3,353 mm) measured from the top of the footing to the centerline of the applied force. The column of 24 in. (610 mm) in diameter was reinforced with 12 No. 8 ($d_b = 25$ mm) deformed bars with a longitudinal reinforcement ratio of 2.08% and transversely confined with No. 4 ($d_b = 12$ mm) spiral deformed bars at 4-in. (102-mm) spacing with a transverse reinforcement ratio of 0.9%. The measured yield strength of the longitudinal and transverse reinforcement bars was 60.6 ksi (418 MPa) and 78.5 ksi (541 MPa), respectively. The concrete compressive strength was 6,340 psi (44 MPa).

Original Column Test to Failure

The original column was tested to failure under an incrementally increasing lateral cyclic load while subjected to a constant axial load of approximately 133 kips (592 kN). The applied axial load simulated the imposed superstructure load and corresponded to approximately 7% of the nominal axial capacity. Three symmetric cycles were applied at each loading stage. Load-displacement hysteresis loops showed significant stiffness degradation and strength loss (75% of peak strength) at the completion of testing. As illustrated in Figure 3, the column failed because of lap splice reinforcement slippage. The damaged state of the column would be classified as extensive or DS-5 for imminent failure–visible reinforcement bars and compressive failure of the concrete core edge (*13, 14*).

Repair Design of the Damaged Column

The repair materials and method were selected to meet the rapid repair requirement. To this end, repair grout, thin sheet metal, and prestressing strands were the only constituent components needed in the developed rapid repair technique.

Materials

A shrinkage-compensating fast-setting repair grout with 1-day strength of 4,500 psi (31 MPa) was used to replace the degraded concrete from the damaged column. The average compressive strength of the repair grout at the test date of the repaired column was determined to be 4,560 psi (32 MPa). The hybrid jacketing was composed of thin cold-formed sheet metal (passive confinement) enclosed by prestressing strands (active confinement). The steel jacketing had the yield strength of 98.5 ksi (680 MPa), tensile strength of 112 ksi (771 MPa), and elastic modulus of 30,050 ksi (207 GPa). According

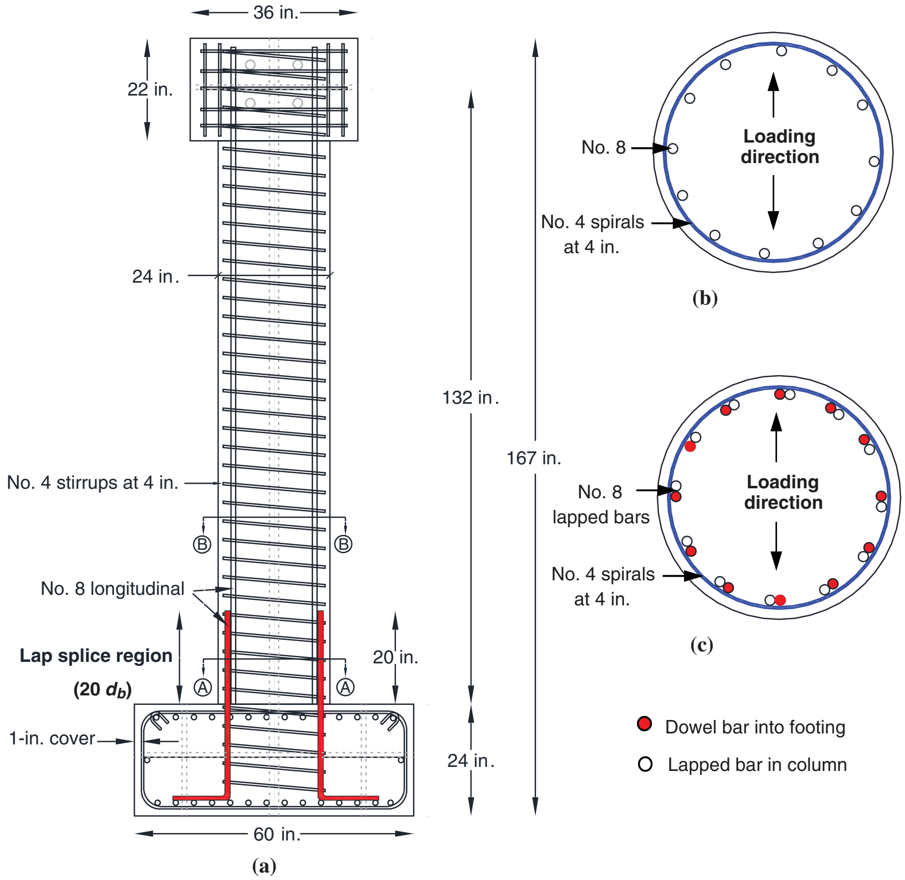


FIGURE 2 Geometry and reinforcement details of the original column: (*a*) original column reinforcement, (*b*) Cross Section B-B, and (*c*) Cross Section A-A [No. 4 (U.S.): No. 13 (SI); No. 8 (U.S.): No. 25 (SI); no. = number].

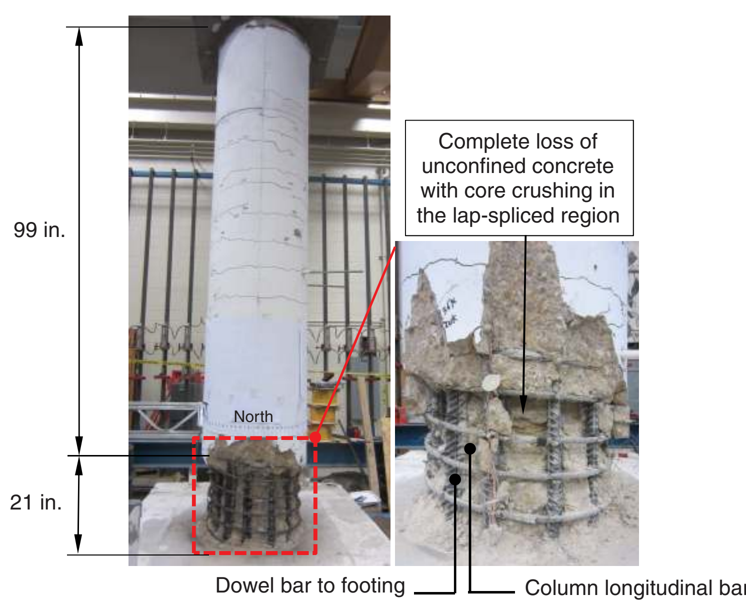


FIGURE 3 Original column damage state.

to the repair design, the sheet metal was 48 in. (1,220 mm) wide and 0.05 in. (1.27 mm) thick. The nominal ½-in. (12-mm) diameter, seven-wire strands had the ultimate tensile strength and modulus of elasticity of 281 ksi (1,937 MPa) and 29,100 ksi (200 GPa), respectively.

Repair Procedure

The damaged column was repaired in its bottom 48 in. (1,220 mm) with the proposed hybrid jacketing. To prevent premature compression damage in the jacketing, a 1-in. (25-mm) gap was left between the steel jacketing and the footing. The entire repair process was completed in four steps as illustrated in Figure 4:

1. Removing the cracked degraded concrete,
2. Placing repair grout to restore the column's original cross section,
3. Wrapping and welding the sheet metal around the column, and
4. Placing and prestressing strands around the steel jacket.

Several special considerations were taken into account in the proposed repair technique. First, existing cracks on the damaged column were not filled by epoxy injection to simulate an emergency repair scenario. Second, while the steel jacket could be directly wrapped on the damaged column and filled with the repair grout, the cross section of the original column was restored before wrapping of the sheet metal with the intent of studying the applicability of this method for bridge retrofit. Third, no surface preparation (such as primer to fill the voids) and adhesive materials (such as epoxy resin to bond the thin sheet metal onto the column) were needed; the proposed method thus requires no curing. Fourth, the force in prestressing strands would remove any gap between the sheet metal and the column. Fifth and last, the proposed repair scheme is similar to FRP jacketing without alteration of the original cross section (no increase in deadweight). Repair grout was allowed to cure for 12 h, and then the lightweight sheet metal and prestressing strands were applied on the damaged column within 4 h.

For real applications of the hybrid jacket on RC bridge columns, when one considers the long-term performance, corrosion resistance, and aesthetic requirements of the rehabilitated column, a protective layer should be provided to the thin steel sheet. For corrosion protection, anticorrosive paintings or coatings could be applied. For aesthetic requirements, a minimum of a 1-in. concrete cover (preferably of high-performance concrete) applied either by shotcreting or by casing concrete over the jacket could be implemented.

Repair Design and Performance Objectives

The column repair was aimed at restoring shear strength and displacement ductility to prevent a lap splice failure of the repaired column. The height of the plastic hinge to be repaired (L_{pz}) was calculated according to California Department of Transportation (Caltrans) seismic provisions (*15*). That is,

$$L_{pz} = \frac{3}{8}\mathrm{AR} \times D \geq 1.5D \qquad (1)$$

where AR is column aspect ratio and D is column diameter. The shear strength of the column was based on the strength of individual components and checked against the factored shear (*13*). That is,

$$\frac{V^o}{\phi} < V_c + V_s + V_j \qquad (2)$$

where

V^o = base shear,
ϕ = 0.85, and
V_c, V_s, and V_j = shear resisted by concrete, existing transverse reinforcement, and jacket, respectively.

Because the existing steel hoop reinforcement in the plastic hinge region yielded during the original column test and no epoxy injection

(a) (b) (c) (d)

FIGURE 4 Repair procedure with the hybrid confining jacket: (*a*) damaged column, (*b*) patched column with repair grout, (*c*) sheet metal wrapping, and (*d*) prestressing strands application.

was applied to repair the concrete cracks, the existing transverse reinforcement and concrete contribution to the shear strength were neglected. The enhanced shear strength attributable to axial load was also neglected since the effect of vertical ground motion may reduce the axial load during an earthquake event (*16*). The hybrid jacketing has two shear-resisting components for the repaired column: thin sheet metal and prestressing strands (*4*). The passive stress contribution of strands was considered in the calculation of shear capacity V_j:

$$V_j = V_{sj} + V_{sp} \tag{3}$$

$$V_{sj} = 0.5\pi t_j f_{yj} D \cot\theta \tag{4}$$

$$V_{sp} = 0.5\pi A_{ps} f_{ps} D s^{-1} \cot\theta \tag{5}$$

where

V_{sj} and V_{sp} = shear enhancement from thin sheet metal and prestressing strands, respectively;
θ = angle of the critical inclined shear-flexure crack to the column axis;
t_j = jacket thickness;
f_{yj} = jacket yield strength;
f_{ps} = level of prestressing stress; and
A_{ps} = cross-sectional area of prestressing strands.

The required confining stress (f_l) to prevent the lap splice failure (*4*) can then be determined by

$$f_l = \frac{A_b f_s}{\mu p l_s} \tag{6a}$$

$$p = \frac{\pi D'}{2n} + 2(d_b + c) \le 2\sqrt{2}\,(d_b + c) \tag{6b}$$

where

A_b = cross-sectional area of nonprestressed reinforcement bar,
f_s = 1.7 times reinforcement yield stress (= 1.7 f_y),
μ = coefficient of friction (assumed as 1.4),
l_s = lap splice length,
p = crack surface perimeter,
n = number of longitudinal lapped bars,
d_b = reinforcement bar diameter,
D' = core diameter (outside-to-outside dimension of the circular transverse reinforcement), and
c = concrete cover thickness.

The maximum transverse strain for the lap splice confinement was limited to 0.001 to prevent splice failure, which is provided by the hybrid jacketing only. The contribution of yielded spirals in repaired columns is negligible (*13, 14*). For the tested column, the required confining pressure would be f_l = 511 psi (3.52 MPa).

The hybrid jacketing was designed to provide sufficient flexural confinement on the damaged column so that the target displacement ductility of 5.0 can be met after repair. The design parameters of the sheet metal [jacket thickness (t_j) and jacket yield strength (f_{yj})] and prestressing strands [level of prestressing stress (f_{ps}), strand spacing (s), and cross-sectional area of prestressing strands (A_{ps})] were then determined accordingly. In this study, the unified energy balanced approach by Mander et al. for confined concrete was adopted to calculate the ultimate achievable strains in the confining jacket (*17*). The confined concrete ultimate compressive strains for the sheet metal and the prestressing strands (*4*) can be determined by

$$\varepsilon_{cu\_sj} = 0.004 + \frac{5.6 t f_{yj} \varepsilon_{sm}}{D f'_{cc}} \tag{7a}$$

$$\varepsilon_{cu\_sp} = 0.004 + \frac{\rho_s f_{pu} \varepsilon_{su}}{f'_{cc}} \tag{7b}$$

where

ε_{cu_sj} and ε_{cu_sp} = ultimate confined concrete strain due to sheet metal and prestressing strands, respectively;
t = steel jacket thickness;
ε_{sm} = strain at peak stress of confining reinforcement;
f'_{cc} = confined concrete compressive strength;
ρ_s = effective volumetric ratio of confining prestressing strands = $4A_{ps}/(D \times s)$;
f_{pu} = ultimate stress level of prestressing strands;
ε_{su} = fracture strain of prestressing strands; and
s = spacing between the prestressing strands.

For the sheet metal and prestressing strands, the effect of lateral confining stress (f_l) on the confined concrete compressive strength (f'_{cc}) can be evaluated by

$$f'_{cc} = f'_c\left[-1.254 + 2.254\sqrt{1 + \frac{7.94 f_l}{f'_c}} - \frac{2 f_l}{f'_c}\right] \tag{8a}$$

$$f'_{cc} = f'_c\left[-1.254 + 2.254\sqrt{1 + \frac{15.88 A_{ps} f_{pu}}{s D f'_c}} - \frac{4 A_{ps} f_{pu}}{s D f'_c}\right] \tag{8b}$$

where

f'_c = specified compressive strength of concrete,
f_l = $(2 \times t \times f_s)/D$, and
f_s = circumferential induced stress in the sheet metal.

After considering the repair performance objectives, shear design, lap splice design, and flexural design with required displacement ductility, the final design of the repaired column is presented in Figure 5. The repair design includes a sheet metal, 0.05 in. (1.27 mm) thick and 48 in. (1,220 mm) wide, and 10 prestressing strands in the plastic hinge region. Specifically, six strands at 4-in. (102-mm) spacing were applied over the lap splice region and the remaining four strands at 6 in. (152 mm) outside the spliced end. Each strand was prestressed up to 14 kips (62 kN) and anchored with a twisted ring anchor from Dywidag-Systems International, Toughkenamon, Pennsylvania.

Instrumentation and Test to Failure of the Repaired Column

Linear variable differential transformers and string potentiometers were used to measure displacement profile, average rotation, and curvature of the column specimen. In addition, the repaired column was instrumented with a total of 110 strain gauges attached on the hybrid jacket (72 strain gauges) and the inside reinforcement

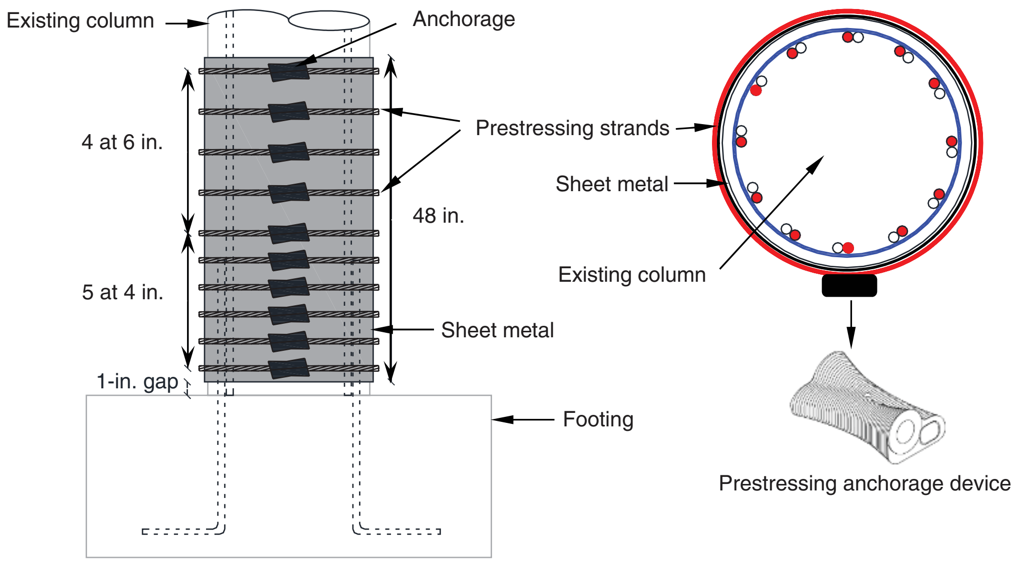


FIGURE 5 Hybrid jacket details.

(38 strain gauges) as illustrated in Figure 6. Six prestressing strands (Nos. 1 to 4, 6, and 8 from the footing) were instrumented with four transverse strain gauges per strand at quarter points. The sheet metal was also instrumented with four transverse strain gauges and four longitudinal strain gauges at quarter points at each of six levels as detailed in Figure 6. Strain gauges on the inside reinforcement before repair were attached to dowel bars and longitudinal and transverse reinforcement bars.

The original and repaired column test setup as shown in Figure 6 was identical. A constant axial load of 133 kips (592 kN) was applied with seven prestressing steel strands through a polyvinyl chloride pipe at the column center. The strands were fixed at the bottom of the footing and at the top of the column loading stub. The axial load was applied with a hydraulic jack and was held constant throughout the test. Each column specimen was laterally loaded through two actuators in incrementally increasing displacement control with three symmetric cycles at each level. The shear force and bending moment of the column when the actuators pushed the column away (south direction) was defined as positive.

RESULTS AND DISCUSSION

The load-drift hysteresis loops and the load-displacement envelopes of the two column specimens are compared in Figure 7, *a* and *b*, respectively. Figure 7*a* shows that the original as-built column specimen exhibited unstable hysteresis loops with rapid strength deterioration and pinched behavior caused by slippage of the lapped

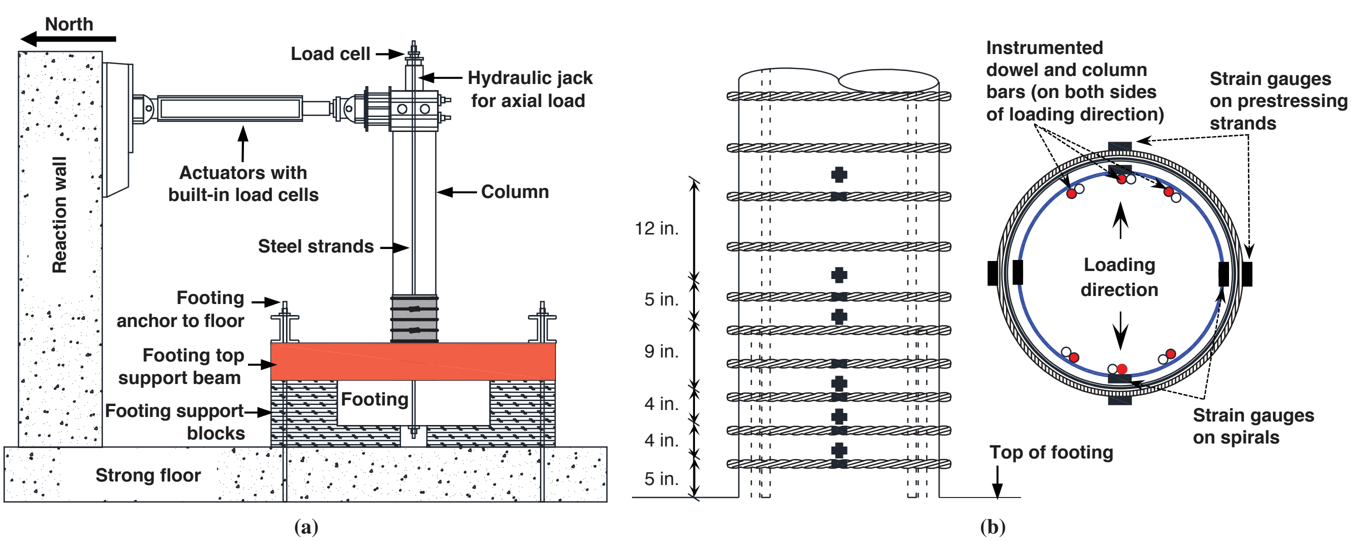


FIGURE 6 Column test setup and instrumentation: (*a*) column test setup and (*b*) strain gauge locations on hybrid jacket and rebars (✚ = bidirectional strain gauge on sheet metal; ▬ = unidirectional strain gauge on strands and spirals).

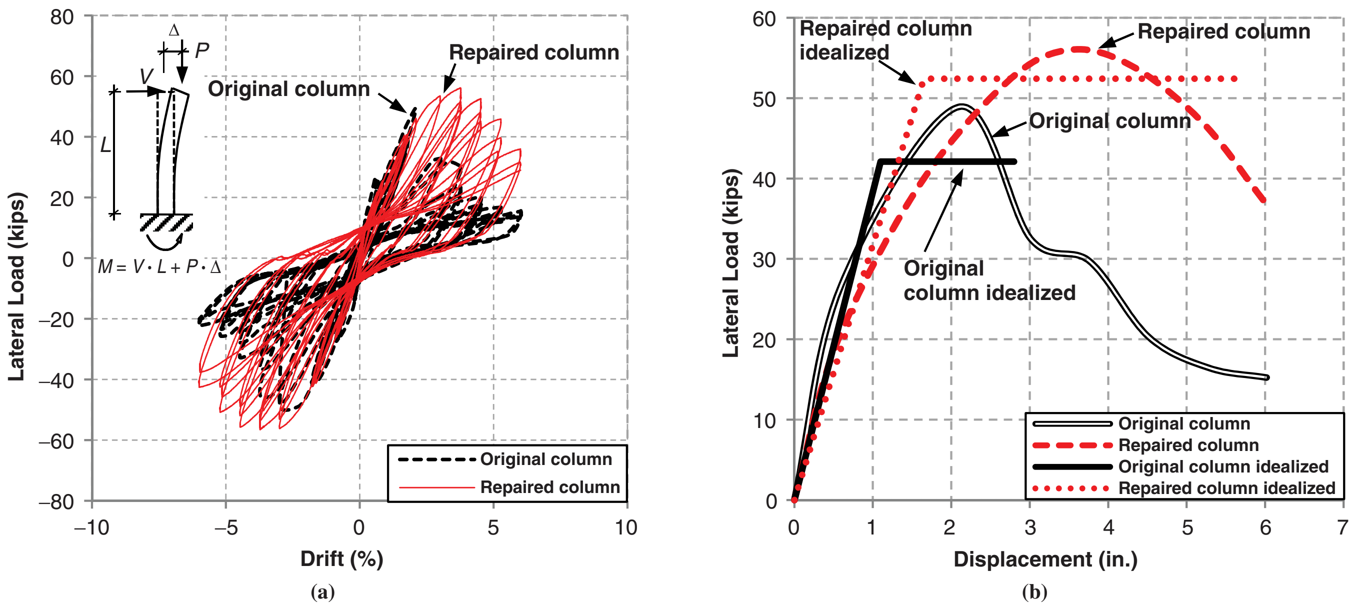


FIGURE 7 Cyclic and idealized force–displacement relationship of original and repaired column specimens: (*a*) hysteresis behavior and (*b*) measured and idealized envelopes (1 kip = 4.45 kN; *M* = moment; *V* = shear force; *L* = column height; *P* = axial force; Δ = lateral displacement).

bars at the column base. Although it reached the nominal flexural strength of the cross section, the original column specimen exhibited rapidly degrading postelastic behavior at large drifts, which is inadequate in seismically active regions.

In comparison with the original column, the repaired column resulted in 15% increase in lateral strength capacity with stable hysteresis behavior. Figure 7*a* indicates that the in-cycle strength degradation of the original column was enhanced to a cyclic strength degradation response of the repaired column. The in-cycle strength degradation can lead to structural collapse under dynamic loading. The repaired column experienced flexural cracks at the column–footing interface at approximately 1.5% lateral drift with the column confinement provided in the plastic hinge. At the same time, the stress in the column was effectively transferred into the footing without lap splice failure, resulting in surface cracks on the footing cover concrete at the location of the dowel bars. The lap-spliced reinforcement reached yielding, as verified by the strain measurements. Further loading enlarged the flexural cracks at the column–footing interface and, under cyclic effect, initiated severe cracking and pinching of the repair grout between the lapped bars. Eventually, crushing of the repaired grout was the governing failure causing the pullout between the dowel and longitudinal bars with no further yielding of the reinforcement.

The sheet metal effectively confined the cover concrete of the repaired column and prevented the concrete cover spalling. It also prevented strand penetration into the cover concrete at large drift angles. Hoop strain measurements indicated that the prestressing strands effectively maintained the confining pressure throughout the column test even though the cover concrete severely cracked.

Repair Efficiency

Strength, stiffness, and ductility capacity were used to assess the structural behavior of the original and repaired column specimens. They were evaluated on the basis of idealized, perfectly elastoplastic load-displacement envelopes as displayed in Figure 7*b* (*12–14*). For the original column, the measured load-displacement envelope was idealized by setting the initial slope to pass through the first longitudinal reinforcement yielding and altering the postelastic region so that areas under the measured and idealized envelopes were equal. For the repaired column, the initial elastic portion of the idealized envelope was acquired by connecting the origin of the measured envelope to a point at which the applied load is equal to one-half of the maximum measured load. The yield level was determined by equating the areas underneath the measured and idealized capacity curves. The three nondimensional response indices for strength, stiffness, and ductility are defined as follows:

Strength index (STRI). The "strength index" is defined as the ratio between the lateral strength of the repaired column (V_r) and that of the original column (V_o):

$$\mathrm{STRI} = \frac{V_r}{V_o} \tag{9}$$

Stiffness index (STFI). The "stiffness index" is defined as the ratio between the service stiffness of the repaired column (K_r) and that of the original column (K_o):

$$\mathrm{STFI} = \frac{K_r}{K_o} \tag{10}$$

Ductility index (DI). The "ductility index" is defined as the ratio between the modified ductility capacity of the repaired column (D_r^*) and that of the original column (D_o):

$$\mathrm{DI} = \frac{D_r^*}{D_o} \tag{11}$$

$$D_r^* = D_r \times \left(\frac{K_o}{K_r}\right) = \frac{D_r}{\text{STFI}} \tag{12}$$

where D_r is ductility capacity of the repaired column.

The ductility capacity is defined as the ratio of the ultimate displacement capacity to effective yield displacement, which can be obtained from the idealized curve (D_r and D_o). The ultimate displacement capacity was defined as the displacement corresponding to 80% of the maximum lateral measured strength. To account for the different initial stiffness of the original and repaired columns, the repaired column ductility was modified as shown in Equation 12.

The three response indexes for the tested columns are presented in Table 1, which shows that the improved seismic behavior of the repaired column over the original column is significant for enhanced strength and ductility. Specifically, the strength of the repaired column is 115% of that of the original column; the ductility of the repaired column is 168% of that of the original column. These results demonstrate the efficacy of the proposed repair method as required in modern seismic codes. However, service stiffness of the repaired column is 83% of that of the original column, mainly because of degraded material properties from the original test. To simulate an emergency postearthquake repair, epoxy injection of cracks in the damaged column was not implemented; thus the repaired column stiffness was not completely restored as compared with that of the original column.

Based on Caltrans seismic provisions (SDC 3.1.4.1) (*15*), a minimum displacement ductility capacity of 3 is required for RC columns. The repaired column could reach a displacement ductility capacity of 4.2 versus 2.5 of the original column (ductility index = 168%). The repair jacket could also restore and even improve the strength of the repaired column versus the original column, satisfying the Caltrans seismic design criteria (*15*). The initial stiffness of the repaired column was 83% of that of the original column, which is in the acceptable range (20% of the original column) for rapidly repaired RC bridge piers (*12–14*).

To compare the hybrid jacket with columns rehabilitated only with active prestressing strands, a retrofitted column from the Beausejour study (*10*) was selected. The selected column had comparable flexural strength capacity compared with the tested column in this study with similar lap splice length (20 d_b). The retrofitted column with prestressing strands exhibited a maximum strength of 47 kips compared with 44 kips of the reference (unretrofitted) column (6.8% strength enhancement). The retrofitted column exhibited rapid strength decay resulting from a loss of confining pressure in strands on penetration of strands into cracked cover concrete. The drift capacity of the retrofitted column was 3% compared with 2% for the reference column (*10*). Therefore, the prestressing confinement alone was less efficient than the proposed hybrid jacket. Also the Beausejour study (*10*) applied prestressing strands into the retrofitting of an existing column (i.e., undamaged and intact column specimen), whereas this study applied the hybrid jacket on a severely damaged column specimen.

TABLE 1 Idealized and Measured Responses of Original and Repaired Column Specimens

Column Specimen Response	Original	Repaired
Idealized Response Values		
Inelastic base shear, kips (kN)	42.1 (187.2)	52.4 (233)
Effective yield displacement, in. (mm)	1.1 (27.9)	1.63 (41.4)
Ultimate displacement, in. (mm)	2.8 (71.12)	5.65 (143.5)
Measured Results		
Lateral strength, kips (kN)	48.91 (217.6)	56.16 (249.8)
Initial service stiffness, kips/in. (kN/mm)	38.27 (6.7)	31.75 (5.63)
Ductility, in./in. (mm/mm)	2.5 (2.5)	4.2 (4.2)

NOTE: Structural response indices: STRI = 115%; STFI = 83%; DI = 168%.

Failure Mode

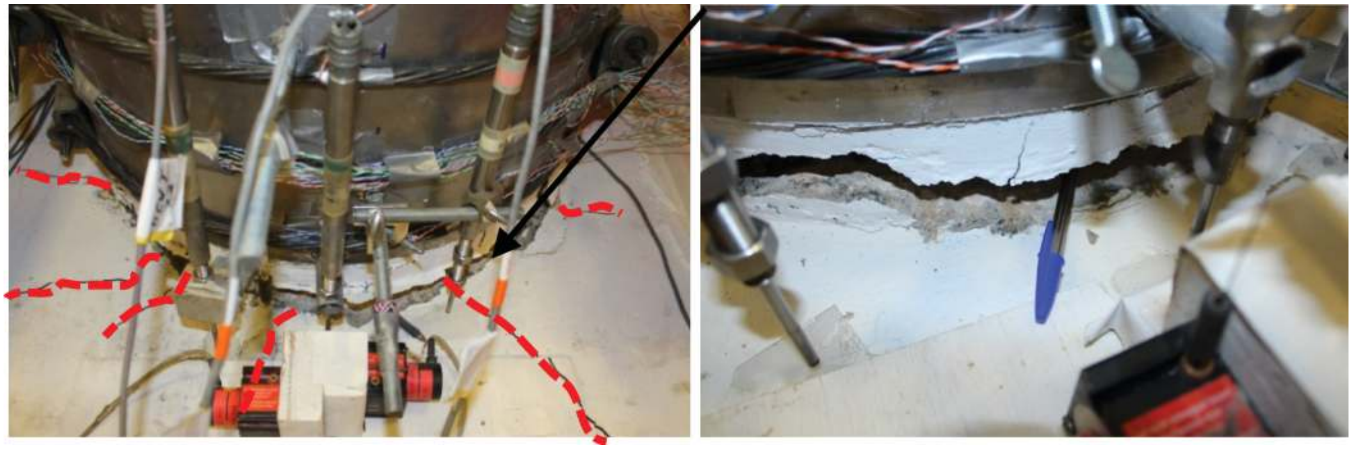
For crack formation, propagation, and distribution in the plastic hinge region, the original and the repaired columns were quite dissimilar. For the original column, the flexural and vertical splitting cracks with limited plastic hinge progression over the height almost equal to the column diameter were the primary observed damage. The cracks mainly resulted from the bond failure of dowel bars at the column–footing connection. No sign of stress transfer from the dowel bars to the footing was observed.

For the repaired column, after the hybrid jacket was removed at the completion of the test, the vertical splitting cracks were found to be limited to the bottom gap. This finding indicated that the hybrid confinement was significant because the vertical splitting cracks did not extend farther up to the column. Cracks initiating from the location of dowel bars were observed on the footing surface, as illustrated in Figure 8, mainly because the hybrid confinement enabled the stress transfer through the dowel bars at the column–footing interface. Although extensive damage through cyclic tests was accumulated at the 1-in. gap between the top of the footing and the bottom of the jacket, the hybrid jacket could effectively prevent splice failure. The most prominent structural feature of the proposed hybrid jacket over other jackets is to resist the shear crack opening in both vertical and horizontal directions (i.e., enhanced aggregate interlock and higher shear strength), while the prestressing strands (similarly with any unidirectional jacket such as FRP wraps) can resist the shear crack opening in the transverse direction only.

CONCLUSIONS

In this study, a new hybrid jacketing technology was developed to rapidly and effectively repair earthquake-damaged bridge columns. Experimental tests on RC column specimens validated the effectiveness of the proposed repair technique. The tested columns were large scale, allowing an investigation of applicability, constructability, and efficacy of the proposed repair method in field conditions. The hybrid jacket requires no heavy installation equipment and no curing time of adhesive material, and therefore it is a viable option for postearthquake emergency repair. On the basis of test results, the following conclusions can be drawn:

- The hybrid confinement is effective in preventing lap splice failure and improving the flexural strength and ductility.
- The proposed hybrid jacketing is lightweight and proves to be applicable as an emergency repair technique for bridge piers.
- The initial stiffness is only partially restored because of the existing damage in the concrete and reinforcement.
- The confining pressure exerted by prestressing strands is adequate for shear transfer between the steel jacketing and column; no adhesive epoxy or dowel reinforcement is required.



(a) (b)

FIGURE 8 Concrete cracks associated with stress transfer through dowel bars at the column–footing connection: (*a*) cracks on footing and (*b*) damage in the plastic hinge (footing cracks marked with red dashed lines).

• Sheet metal provides the required bearing strength and prevents cover concrete spalling and penetration of the prestressing cables.

• Prestressing cables could sustain the active confining pressure up to 6% lateral drift with no significant prestressing force loss.

• The proposed repair method is straightforward and the repair design equations can be readily used in practical applications.

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REFERENCES

1. DeBlasio, A. J., A. Zamora, F. Mottley, R. Brodesky, M. E. Zirker, and M. Crowder. *Effects of Catastrophic Events on Transportation System Management and Operations, Northridge Earthquake—January 17, 1994.* U.S. Department of Transportation, 2002.
2. *United States Geological Survey.* 2012. http://www.usgs.gov. Accessed May 2014.
3. *Seismic Retrofitting Manual for Highway Structures: Part 1, Bridges.* Report FHWA-HRT-06-032. FHWA, U.S. Department of Transportation, 2006.
4. Priestley, M. J. N., F. Seible, and G. M. Calvi. *Seismic Design and Retrofit of Bridges.* John Wiley & Sons, Inc., New York, 1996.
5. Lin, Y., W. L. Gamble, and N. M. Hawkins. *Testing of Bridge Piers, Poplar Street Bridge Approaches, East St. Louis, IL.* Department of Civil Engineering, University of Illinois at Urbana–Champaign, 1994.
6. Saiidi, M. S., and N. Mangoba. Seismic Retrofit of Flared Reinforced Concrete Columns in Multispan Viaduct: From Research to Field Implementation. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1892,* Transportation Research Board of the National Academies, Washington, D.C., 2004, pp. 189–196.
7. Walkenhauer, B. J., D. I. McLean, C. Boone, and M. Lamont. Seismic Retrofitting of Cruciform-Shaped Columns by Jacketing with Fiber-Reinforced Polymer. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2251,* Transportation Research Board of the National Academies, Washington, D.C., 2011, pp. 135–143.
8. Fakharifar, M., M. K. Sharbatdar, Z. Lin, A. Dalvand, A. Sivandi-Pour, and G. Chen. Seismic Performance and Global Ductility of RC Frames Rehabilitated with Retrofitted Joints by CFRP Laminates. *Earthquake Engineering and Engineering Vibration,* Vol. 13, No. 1, 2014, pp. 59–73.
9. Fakharifar, M., G. Chen, Z. Lin, and Z. Woolsey. Behavior and Strength of Passively Confined Concrete Filled Tubes. *Proc., 10th U.S. National Conference on Earthquake Engineering,* Anchorage, Alaska, 2014.
10. Beausejour, P. *Seismic Retrofit of Concrete Columns with Splice Deficiencies by External Prestressing.* MS thesis. University of Ottawa, Ontario, Canada, 2000.
11. Fakharifar, M., A. Dalvand, M. Arezoumandi, M. K. Sharbatdar, G. Chen, and A. Kheyroddin. Mechanical Properties of High Performance Fiber Reinforced Cementitious Composites. *Construction and Building Materials,* Vol. 71, 2014, pp. 510–520.
12. Fakharifar, M., G. Chen, L. Sneed, and A. Dalvand. Seismic Performance of Post-Mainshock FRP/Steel Repaired RC Bridge Columns Subjected to Aftershocks. *Composites Part B: Engineering,* Vol. 72, 2015, pp. 183–198.
13. Vosooghi, A., and M. S. Saiidi. Design Guidelines for Rapid Repair of Earthquake-Damaged Circular RC Bridge Columns Using CFRP. *Journal of Bridge Engineering,* Vol. 18, No. 9, 2012, pp. 827–836.
14. Vosooghi, A., and M. Saiidi. *Post-Earthquake Evaluation and Emergency Repair of Damaged RC Bridge Columns Using CFRP Materials.* Report CCEER-10-05. Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, 2010.
15. *Seismic Design Criteria (SDC), Version 1.4.* California Department of Transportation, Sacramento, 2006.
16. Higazy, E. M. M., A. S. Elnashai, and M. S. Agbabian. Behavior of Beam-Column Connections Under Axial Column Tension. *Journal of Structural Engineering,* Vol. 122, No. 5, 1996, pp. 501–511.
17. Mander, J. B., M. J. N. Priestley, and R. Park. Theoretical Stress-Strain Model for Confined Concrete. *Journal of Structural Engineering,* Vol. 114, No. 8, 1988, pp. 1804–1826.

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