Executive Summary

Emergency Repair of Damaged Bridge Columns Using Fiber Reinforced Polymer (FRP) Materials

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Executive Summary

1. Introduction

The current seismic design practice in bridge engineering for standard bridges allows for damage to bridge columns during moderate and strong earthquakes. The target response under the maximum credible earthquake is “no-collapse,” realizing that the structure would undergo considerable nonlinearity associated with extensive concrete damage, yielding of bars, or even rupture of some of the bars. For the more frequent earthquakes, the target response is repairable damage that would allow for relatively rapid restoration of the bridge and the highway network. The level of damage to different columns of a bridge varies depending on the intensity of the ground shaking, type of earthquake, and the force/deformation demand on individual members. Based on the inspection of the damaged columns engineers have to determine whether the bridge is sufficiently safe to be kept open to traffic without repair, whether it is repairable within a reasonable time frame, or if it needs to be replaced. Engineers should also recommend repair methods for the columns. Any delay in opening the bridge to traffic can have severe consequences on the passage of emergency vehicles, detour lengths, and traffic congestion in the area. Rapid and effective repair methods are needed to enable quick opening of the bridge to minimize impact on the community.

The present research was aimed at developing guidelines for reliable and efficient repair procedure of earthquake-damaged reinforced concrete (RC) columns using carbon fiber reinforced polymers (CFRPs). This study was composed of three main parts: (1) development of simple criteria for apparent damage states and correlating them to seismic response parameters, (2) experimental and analytical studies of original and repaired column models, and (3) development of rapid repair design recommendations.

2. Damage States and Response Parameters

In the first part of the study, detailed data from 33 bridge column models, mostly tested on shake tables, were evaluated to determine the correlation between the apparent damage and seismic response parameters. Five distinct damage states were proposed based on the apparent damage. The damage states were flexural cracks (DS-1), first spalling with possible shear cracks (DS-2), extensive cracks and spalling (DS-3), visible lateral and/or longitudinal bars (DS-4), and start of core damage indicating imminent failure of the column (DS-5). The damage states are shown in Fig. 1.

Six response parameters were defined in terms of drift, frequency, strains, and yield and ultimate displacements. The response parameters were the maximum drift ratio during the earthquake (MDR), residual drift ratio after the earthquake (RDR), the ratio of the fundamental frequency of a column during the earthquake to its initial uncracked fundamental frequency (FR), the maximum strain in longitudinal steel (MLS), the maximum strain in transverse steel (MTS), and the inelasticity index (II) defined as follows:

$$\begin{align*}
II &= \frac{D_{\text{max}} - D_y}{D_u - D_y} \\
&= \frac{D_{\text{max}} - D_y}{D_u - D_y}
\end{align*}$$

(1)
Where, $D_{max}$ = the maximum lateral displacement of the column during the test leading to a given damage state, $D_u$ = the ultimate (failure) displacement of the column, and $D_Y$ = the effective yield displacement of the column.

Two approaches were used to correlate the damage states and the response parameters. In the first approach, the correlation was identified for five different column categories based on
seismic design, shear demand, and ground motion type. In the second approach, all the columns were studied as a single group to facilitate the use of final recommendations. To consider the effect of data scatter, a probabilistic approach of “fragility function” was used and fragility curves were developed to correlate the damage states and the response parameters. Figure 2 shows the fragility curves for the response parameters at each damage state.

Figure 2. Fragility curves

3. Experimental Studies

In the second part of the study, one standard bent with two low shear columns (Bent-2), two standard high shear columns (NHS1 and NHS2), one low shear substandard column (OLS), and one high shear substandard column (OHS) were tested on the shake table, repaired using CFRP, and retested on the shake table to evaluate the repair performance. In the following paragraphs, the column models are briefly discussed.
### 3.1. Two-Column Bent

Choi et al. (2007) tested a ¼ scale two-span-bridge model on three shake tables at the University of Nevada, Reno. The bridge was subjected to synthetic earthquake records simulating fault rupture. The seismic design of the bridge was based on current seismic codes (Johnson et al. 2006). Grade 60 steel and concrete compressive strength of 5 ksi [34.5 MPa] were specified for the design. Because the middle bent (Bent-2) was the most severely damaged pier, it was used in the current emergency repair study. The bent details are shown in Fig. 3 and the basic properties of the columns are listed in Table 1. The axial load index is defined as the compressive axial force due to gravity loads divided by the product of the cross section area of the column and the specified concrete compressive strength. This index varies typically from 5% to 10% in bridge columns and it was 8.2% in the 2-span bridge model. The average shear stress is calculated as the ratio of the plastic shear divided by the effective shear area. The effective shear area is taken as 80% of the gross section area ($A_g$) in circular columns. The shear stress index is calculated by dividing the average shear stress by $\sqrt{f'_c}$ [psi] or $0.083\sqrt{f'_c}$ [MPa]. This index is used to determine the level of shear stress in columns. In this project, the index smaller than 4 was selected as the low shear level in the column.

The columns in the bent were at damage state five at the end of the shake table tests. Under this damage state, many spirals and longitudinal bars are visible, some of the longitudinal bars slightly buckle, and the edge of concrete core is damaged. Note that there is no bar fracture at this damage state.

![Figure 3. Details of Bent-2](image)

**Table 1. Column basic properties**

<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>Bent-2</td>
<td>96</td>
<td>2438</td>
<td>156%</td>
<td>0.9%</td>
<td>4</td>
<td>46.5</td>
<td>206.8</td>
</tr>
<tr>
<td>NHS1, NHS2</td>
<td>80</td>
<td>2032</td>
<td>16</td>
<td>3.08%</td>
<td>1.38%</td>
<td>25</td>
<td>100</td>
</tr>
<tr>
<td>OLS</td>
<td>80</td>
<td>2032</td>
<td>16</td>
<td>1.59%</td>
<td>0.14%</td>
<td>5.0</td>
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<tr>
<td>OHS</td>
<td>80</td>
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<td>16</td>
<td>2.47%</td>
<td>0.14%</td>
<td>2.5</td>
<td>60</td>
</tr>
</tbody>
</table>

### 3.2. NHS1 and NHS2

As part of this research, two similar high shear standard RC bridge columns were designed, constructed and tested on one of the shake tables at the University of Nevada, Reno to
reach damage state 5. NHS1 and NHS2, new-design high shear, are the designations of the two identical column models tested under double-curvature bending (Fig. 4). This configuration allows for the application of a relatively high shear. The repaired columns were labeled NHS1-R and NHS2-R, respectively. NHS1 and NHS2 were designed and constructed at different times because the study of NHS2 was found to be necessary after NHS1-R testing and analysis of the data. The latest Caltrans Seismic Design Criteria, SDC version 1.4 (Caltrans 2006), was used to design the columns, and the basic properties of the columns were listed in Table 1.

![Figure 4. Details of the shake table double-curvature bending setup](image)

3.3. OLS and OHS

Reinforced concrete bridge columns designed prior to the 1970’s were not adequately detailed to resist seismic loads and are considered to be sub-standard. They have inadequate lateral reinforcement and their longitudinal bars are lap spliced at the base. The common failure modes in sub-standard columns are shear, bond degradation in the lap-splice zone, premature concrete failure due to lack of confinement, or a combination of these failure modes. The dominant failure mode in sub-standard high shear columns is shear failure, which is completely brittle, and that of sub-standard low shear columns is lap-splice failure with a limited ductility. Since the lap-splice degrades gradually, some yielding could occur before failure in low shear columns.

As part of this research, one substandard low shear and one substandard high shear RC bridge columns were designed, constructed and tested on one of the shake tables at the University of Nevada, Reno. They were tested to reach the highest repairable damage state, but to avoid total failure. OLS was tested to reach damage state 3 (DS-3). DS-3 consists of extensive spalling and shear cracks. OHS was tested to reach damage state 2 (DS-2). DS-2 consists of minor spalling and might include minor shear cracks. Due to the severely inadequate transverse steel and longitudinal bar lap-splice at the column base, the original substandard columns could not reach higher damage states. After the tests, the columns were repaired using a CFRP jacket and were subsequently tested on the shake table to evaluate the performance of the repair.

OLS and OHS are the designations used for the old-design low shear and the old-design high shear column models, respectively. The low shear column was tested under single-
curvature bending (Fig. 5), and the high shear column was tested under double-curvature bending setup (Fig. 4). The repaired columns were labeled OLS-R and OHS-R, respectively. Table 1 summarizes the basic properties of the sub-standard column models.

![Figure 5. Details of the shake table cantilever bending setup](image)

3.4. Repair Design

The repair of the standard column models was designed with the objective of restoring the lateral load and ductility capacity of the column. A unidirectional carbon fiber reinforced polymer (CFRP) jacket was used for this purpose. Other types of jacket were not considered because they were beyond the scope of this study. Different contribution percentages of concrete and spirals shear strength were used. The columns of Bent-2 were the first columns tested, repaired, and retested. Due to lack of information about the contribution of the concrete and spirals to the shear strength of the repaired columns, their contributions were neglected conservatively along the entire column height. Because there are no seismic repair design guidelines, seismic retrofit guidelines of Caltrans Memo to Designer 20-4, attachment B, MTD 20-4 (Caltrans 2007), were used to restore confinement using CFRP jacket. This document requires providing a confinement pressure of 300 psi [2.07 MPa] at a radial dilating strain of 0.004 in the plastic hinge regions. The resulting jacket consisted of two layers of CFRP at the plastic hinge regions and one layer of CFRP elsewhere.

Inside the plastic hinge region, since some of the thin cracks are not repairable inside the core, shear strength of concrete was neglected in NHS1-R and NHS2-R. The spirals of NHS1 experienced a strain greater than 1.6 times the yield strain. As a result, the shear strength of the spirals was assumed to be zero for NHS1-R. Subsequent to testing the strain data indicated that NHS1-R spirals and CFRP jacket contributed to the shear strength equally even though spirals were neglected in design. A second column, NHS2-R was designed neglecting the concrete shear strength, and the jacket was designed for one-half the column shear demand rather than providing the total shear strength. Outside the plastic hinge region, since the spirals did not yield, 100% of spirals shear strength was used for NHS1-R and NHS2-R. Although shear cracks occurred outside the plastic hinge as well, the level of damage was much lower than that of the
plastic hinge. As a result, 50% of the concrete shear strength was assumed to exist. The provisions of the Caltrans Memo to Designer 20-4, attachment B, MTD 20-4 (Caltrans 2007), were used to restore confinement using CFRP jacket.

In NHS1-R, the jacket consisted of four layers of CFRP at plastic hinge regions and one layer of CFRP elsewhere. In NHS2-R, the jacket consisted of two layers of CFRP at plastic hinge regions and one layer of CFRP elsewhere.

The repair of substandard column models was designed with the objective of restoring and upgrading shear strength, preventing lap-splice slippage, and upgrading confinement of the columns by using unidirectional CFRP jacket. The existing steel hoop contribution to the shear strength was neglected in the repaired substandard column models. Inside the plastic hinge region, since some of the thin cracks were not repairable, the shear strength of the concrete was also neglected. Although shear cracks occurred outside the plastic hinge as well, the level of damage was much less than that of the plastic hinge. As a result, 50% of the concrete shear strength was counted on in the repair design.

Priestly et al. (1996) showed that propensity for splice failure could be predicted by an assessment of the concrete tensile capacity across a potential splitting failure surface. After cracking develops on this surface, splice failure can be inhibited if adequate clamping pressure is provided across the fracture surfaces by confinement. A friction coefficient of 1.4 is appropriate if the equivalent radial dilation strain is less than 0.0015. Priestly et al. (1996) used this provision to design the lap-splice retrofit. Because there were no seismic repair design guidelines, the retrofit provision was used to inhibit lap-splice failure. Due to reason discussed previously, seismic retrofit guidelines of Caltrans Memo to Designer 20-4, attachment B, MTD 20-4 (Caltrans 2007), were used to design for confinement provided by the CFRP jacket.

The jacket for OHS-R consisted of two layers of CFRP along the entire column height and the jacket for OLS-R consisted of two layers of CFRP at the plastic hinge region and one layer of CFRP outside the plastic hinge region.

3.5. Repair Procedure

The repair procedure consisted of straightening the column, removing the loose concrete, epoxy injection of cracks, concrete repair using a fast-set, non-shrink mortar, CFRP wrapping, and accelerated curing of CFRP under elevated temperature for 24 hours followed by a day of curing under ambient laboratory temperature. The total repair and curing time was three to four days, including two days of curing for CFRP. Figure 6 shows the details of the rapid repair procedure.
3.6. Repair Performance

Strength, stiffness, and drift capacity are the key performance parameters that indicate the adequacy of a structure under seismic loading. To evaluate the performance of the repair, these parameters for the repaired columns were compared with those of the original columns. The elasto-plastic idealized envelopes were used to estimate the strength and stiffness of the column models.

The drift capacity of the original bent (Bent-2) was estimated at 14% and that of the repaired bent (Bent-2R) was measured at 13.1%. Therefore, it can be concluded that the drift capacity of the bent was nearly restored by the repair. Using the elasto-plastic idealized force-displacement relationship, the repaired bent capacity was 1.1% higher than that of the original bent. As a result, the bent capacity was fully restored by the repair. The initial slope of the idealized curve was used to indicate the initial stiffness of the bent. Based on the idealized curves, the repaired bent initial stiffness was 70.8% of that of the original bent. Consequently,
the stiffness could not be restored completely by the repair because of material stiffness
degradation during the original runs and the fact that some of the microcracks could not be fully
injected with epoxy resin. Figure 7 shows the measured envelopes and idealized elasto-plastic
envelopes for the bent.

![Figure 7. Measured (left) and idealized elasto-plastic envelopes (right) for Bent 2](image)

The drift capacity of NHS1 and NHS2 was estimated at 9.0% and 7.7%, and that of
NHS1-R and NHS2-R was measured at 13.1% and 13.3%, respectively. Consequently, the drift
capacities of the columns were fully restored by the repair. The higher drift capacity of the
repaired columns was due to the higher confinement provided by the jacket and spirals than that
of original columns provided solely by spirals. Using the idealized envelopes, NHS1-R strength
was equal to that of NHS1, and NHS2-R strength was 13% higher than that of NHS2. As a
result, the columns strength was fully restored by the repair. The initial slope of the idealized
curve was defined as the column stiffness. Based on the idealized curves, NHS1-R stiffness was
38% of that of NHS1 and NHS2-R stiffness was 56% of that of NHS2. Consequently, the
stiffness could not be restored completely by the repair; however, the concrete repair method
and the material used in NHS2-R were more effective in restoring the column stiffness than those of
NHS1-R. Figure 8 shows the measured envelopes and idealized elasto-plastic envelopes for the
columns.
Figure 8. Measured and idealized elasto-plastic envelopes for columns

Figure 8 shows that the confinement and shear strength of the substandard columns were upgraded effectively so that the repaired columns underwent a reasonable plastic deformation.
before failure. The idealized curves were used to estimate the strength, stiffness, and the ductility of the substandard columns. In both columns the strength was increased after repair. The strength of OLS-R was increased by 12% and that of OHS was increased by 42%. The stiffness of OLS-R was 70% of that of OLS and the stiffness of OHS-R was 90% of that of OHS.

In standard column repair, restoring the drift capacity of the original column was one of the repair objectives. Since the substandard columns did not meet the current seismic requirements, increasing the drift capacity was also considered as a repair objective. The displacement ductility capacity, \( \mu_c \), was used as a parameter to evaluate the drift capacity of the columns and was calculated as the ratio of ultimate displacement to the effective yield displacement. Using the idealized force-displacement curves for the repaired columns, the displacement ductility capacity was calculated as 3.13 and 3.54 for OLS-R and OHS-R, respectively. Even though these values are smaller than the target displacement ductility of 5, they are larger than the minimum required displacement ductility capacity of 3 in SDC 3.1.4.1 for modern columns. Due to longitudinal bars yielding during the original tests, the initial stiffness of the repaired columns was smaller than those of the original columns and consequently the effective yield displacement was larger. As a result, another estimate of the displacement ductility capacity of the repaired columns was made based on the stiffness related to the first yield point of the original columns. The modified displacement ductility capacity of \( \mu'_c \) is defined as follows:

\[
\mu'_c = \frac{(\Delta_\text{c})_R}{(\Delta_\text{Y})_R} \times \frac{K_O}{K_R}
\]  

(2)

Where \((\Delta_\text{c})_R\) and \((\Delta_\text{Y})_R\) are the ultimate and the effective yield displacement of the repaired column, and \(K_O\) and \(K_R\) are the stiffness of the original and repaired column, respectively. Note that all parameters are calculated based on the idealized elasto-plastic curves. The modified displacement ductility capacity of 4.45 and 3.93 was calculated for OLS-R and OHS-R, respectively. These values are still smaller than the ductility target of 5, but they satisfy the minimum required ductility of 3 in SDC 3.1.4.1 (Caltrans 2006).

### 4. Analytical Studies

The Open System for Earthquake Engineering Simulation (OpenSees) software was utilized for analytical studies (OpenSees 2006). Routine modeling techniques were used for the original columns to determine the accuracy of the techniques in calculating the nonlinear response of the reinforced concrete bridge columns. However, for repaired columns a new modeling method had to be developed to account for prior damage to different damage states. In the analyses, bond-slip deformations, shear deformations, strain rate effects on material properties, and P-Delta effect were included. The columns were modeled with nonlinear beam/column element discretized by uniaxial fiber elements. In fiber element method, equilibrium between external forces and fibers forces and compatibility among fibers deformation need to be satisfied. The fibers shorten or elongate so that plane sections remain plane after deformation.
4.1. Original Columns

Pushover and nonlinear dynamic analyses were conducted for the original columns. The measured and calculated pushover curves for Bent-2 and NHS2 are shown in Fig. 9 and cumulative force-displacement hysteresis curves for NHS1 and NHS2 are shown in Fig. 10. Good correlation can be observed between the calculated and measured data in terms of stiffness and strength. This proves the accuracy of the applied techniques in calculating the nonlinear response of the original column models.

![Figure 9. Measured and calculated pushover curves for NHS2 and Bent 2](image)

![Figure 10. Cumulative force-displacement hysteresis curves for NHS1 and NHS2](image)

4.2. Repaired Columns

As discussed previously, the columns stiffness was not fully restored by the repair. Therefore, standard analysis methods could not be used directly for repaired columns, and a new method had to be developed. In the proposed method, the steel properties were modified to account for column softening due to earthquake damages. In addition, existing equations to estimate the column shear stiffness were modified to account for CFRP jacket contribution. Bond-slip deformations, strain rate effects on material properties, and P-Delta effect were included in the analyses.

4.2.1. Modification of Steel Properties

Under cyclic loading the stress-strain properties of steel become different from those associated with purely tensile or compressive stress. This is known as the Bauschinger effect (Kent & Park 1973) and results in lowering of the reversed yield stress and the reversed stiffness.
Consequently, reversed loading curves are important when considering the effects of high-intensity seismic loading on members.

The longitudinal bars of the repaired columns underwent cyclic loading during the original shake table tests. Therefore, the steel properties of the repaired columns had to be modified to account for the Bauschinger effect. A tri-linear stress-strain relationship was proposed for the repaired columns longitudinal bars (Fig. 11). The slope of the first branch was calculated as a fraction of the steel modulus of elasticity. The modification factor ($\alpha$) was determined based on the damage state. For instance, the factor of 0.2, 0.5, and 0.67 were proposed for DS-5, DS-3, and DS-2, respectively. Point A represented the yield stress and the strain associated with the modified stiffness. The yield stress was modified to account for strain rate effect using the magnification factors calculated in previous sections. The second branch connected Point A to Point B. Point B was related to the maximum strain in longitudinal steel (MLS) at the given damage state. The third branch connected Point B to the ultimate point (Point C). The stresses corresponding to Point B and C were modified to account for strain rate effect.

![Figure 11. Original and modified stress-strain relationship for steel](image)

4.2.2. Pushover Analysis

Pushover analyses were conducted for the repaired columns using OpenSees software (OpenSees 2006). Pushover analyses were conducted for two cases. In the first case it was assumed that the columns were retrofitted; therefore, original steel properties were assigned to the models. In the second case it was assumed that the columns were repaired; therefore, the modified steel properties were assigned to the models. Sample results are plotted in Fig. 12. It can be seen that by modifying the steel element properties very close correlation was obtained between the measured and calculated results.
5. Column Repair Design Guidelines

In the third part of the study, a design procedure for unidirectional carbon fiber reinforced polymer (CFRP) jacket was developed with the objective of restoring shear strength and confinement of earthquake-damaged standard bridge columns to provide a displacement ductility capacity of 5. The proposed design procedure for CFRP jacket is described in the following sections.

5.1. Restoring Shear Strength

The shear strength of the CFRP jacket and the required jacket thickness were calculated using Equation 3 and 4:

\[
V_j = \frac{V_o}{\phi} - (R_c V_c + R_s V_s) \quad (3)
\]

\[
t_j = \frac{V_j}{\pi/2 \varepsilon_j E_j D} \quad (4)
\]

Where \(V_j\) is the required shear strength of the jacket, \(R_c\) and \(R_s\) are the contribution ratios of concrete and spirals to the shear strength, respectively, \(\phi = 0.85\), \(V_o\) is the overstrength shear,
$V_c$ is the concrete shear strength, and $V_s$ is the spirals shear strength. $V_o$, $V_c$, and $V_s$ can be determined based on Caltrans SDC (2006).

In Equation 4, $t_j$ is the required jacket thickness, $\varepsilon_j$ is the dilating strain of the jacket, $E_j$ is the jacket modulus of elasticity, and $D$ is the column diameter. Priestley et al. (1996) recommended that in calculating the shear resistance contributed by the FRP jacket, the radial dilating strain of FRP ($\varepsilon_j$) be limited to 0.004 to avoid degradation in concrete aggregate interlock.

During experimental studies, for target displacement ductility of 5, the measured $R_c$ was 0.6, 0.7, 0.8, 0.9, and 1.0 for a plastic hinge under DS-5, DS-4, DS-3, DS-2, and DS-1, respectively. Since the concrete contribution to shear in the repaired plastic hinges is related to the quality of the epoxy injection, and the injection quality in the field is expected to be weaker than the laboratory conditions, $R_c$ is recommended to be 20%, 40%, 60%, 80%, and 100% for plastic hinges under DS-5 to DS-1, respectively. Since the importance of epoxy injection is increased with damage state, the safety factor applied to $R_c$ was increased by damage state. For instance, a safety factor of 3 was applied to $R_c$ at DS-5, and a safety factor of 1.125 was applied to $R_c$ at DS-1.

During experimental studies, for target displacement ductility of 5, $R_s$ was measured at 61.4% for a plastic hinge under DS-4 or DS-5. Conservatively, $R_s$ is recommended at 50% for plastic hinges under DS-4 or DS-5. Since $R_s$ was not measured for the remaining damage states, it was estimated for DS-1 to DS-3 based on judgment. A contribution of 100% to the shearing strength was assumed for the spirals which remained elastic during an earthquake. No spiral yielding was expected under DS-1 to DS-3. Consequently, $R_s$ was assumed to be 1.0 for DS-1 and DS-2 and a transition ratio of 0.75 was used for DS-3.

During experimental studies, it was shown that the shear force carried by concrete and spirals outside the plastic hinges does not change significantly after the repair. Consequently, $R_s$ is recommended to be 1.0 for outside the plastic hinges under all damage states and, $R_c$ is recommended at 1.0 for the first three damage states and conservatively, 0.8 and 0.6 for DS-4 and DS-5, respectively.

### 5.2. Restoring Confinement

During experimental studies, the required confinement pressure by CFRP jacket to provide different displacement ductility capacities was measured. A confinement pressure of 659 psi [4544 kPa] was measured in plastic hinges under DS-4 or DS-5 to provide a displacement ductility of 5. This pressure should be provided at the ultimate strain of the jacket. Conservatively, the confinement pressure of 750 psi [5171 kPa] is recommended at the ultimate strain of the jacket for the plastic hinges under DS-4 or DS-5. Assuming an ultimate strain of 0.01 for the CFRP jacket, the recommended confinement pressure would be 300 psi [2068 kPa] for a jacket nominal design strain of 0.004. This pressure matches the requirement of Caltrans (2007) for seismic retrofit.

During experimental studies, confinement pressures of 543 psi [3474 kPa] and 573 psi [3951 kPa] were measured in a plastic hinge under DS-3 and DS-2, respectively, to provide a displacement ductility of 5 in the repaired column. This pressure should be provided at the ultimate strain of the jacket. Conservatively, a confinement pressure of 575 psi [3965 kPa] is recommended at the ultimate strain of the jacket for the plastic hinge zones with DS-2 or DS-3.
No confinement restoration is recommended for non-plastic hinge regions and plastic hinges under DS-1. The required CFRP jacket thickness can be calculated as follow:

$$t_j = \frac{f_iD}{2E_j\varepsilon_{ju}}$$  \hspace{1cm} (5)

Where $t_j$ is the jacket thickness, $f_i$ is the recommended confinement pressure, $D$ is column diameter, $E_j$ is the jacket modulus of elasticity, and $\varepsilon_{ju}$ is the ultimate dilating strain of the jacket.

Table 2 lists the recommended ratio of concrete and spirals contribution to the shear strength and the recommended confinement pressures for all damage states. It can be seen that no restoration is required for a column under damage state 1 (DS-1).

<table>
<thead>
<tr>
<th>Plastic Hinge Zone</th>
<th>Non-Plastic Hinge Zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contribution to Shear</td>
<td>Confinement Pressure for CFRP</td>
</tr>
<tr>
<td>$R_c$</td>
<td>$R_s$</td>
</tr>
<tr>
<td>DS-1</td>
<td>100%</td>
</tr>
<tr>
<td>DS-2</td>
<td>80%</td>
</tr>
<tr>
<td>DS-3</td>
<td>60%</td>
</tr>
<tr>
<td>DS-4</td>
<td>40%</td>
</tr>
<tr>
<td>DS-5</td>
<td>20%</td>
</tr>
</tbody>
</table>

5.3. CFRP Jacket Design for Sub-Standard Columns

The CFRP jacket was designed with the objective of restoring and upgrading shear strength, preventing lap-splice slippage, and upgrading confinement of substandard columns. These objectives are described in the following sections.

5.3.1. Restoring and Upgrading Shear Strength

The required CFRP jacket thickness is determined using Equation 3 and 4. Due to the very low amount of transverse steel in sub-standard columns, $R_s$ is assumed to be zero inside and outside the plastic hinge regions.

During experimental studies, $R_c$ was measured at 69.1% and 30.5% for the plastic hinge zones in substandard columns under DS-2 and DS-3, respectively. Conservatively, $R_c$ is recommended at zero for plastic hinge regions and 0.5 for non-plastic hinge regions.

5.3.2. Inhibiting Lap-Splice Failure

Experimental studies indicated that the retrofit design criteria for inhibiting lap-splice failure developed by Priestley et al. (1996) was sufficient for seismic repair design, as well. Therefore, these criteria are recommended for repair of substandard columns to prevent lap-splice failure. The confining stress necessary to inhibit lap-splice failure of a lapped bar is as follow:
\[ f_i = \frac{A_b f_s}{\mu p l_s} \]  

(6)

Where \( A_b \) is the bar area, \( f_s \) is the transfer stress, \( \mu \) is coefficient of friction, \( l_s \) is the splice length and \( p \) is the perimeter of the crack surface. \( f_s \) is recommended to be 1.7 times the expected yield stress to account for strain hardening and strain rate effects. The coefficient of friction is recommended to be 1.4.

The perimeter of the crack surface for circular columns is calculated as follows:

\[ p = \frac{\pi D^n}{2n} + 2(d_b + c) \leq 2\sqrt{2}(d_b + c) \]  

(7)

Where \( n \) is the number of longitudinal bars of diameter \( d_b \) evenly spaced around the core of diameter \( D^n \) with concrete cover of \( c \).

Based on confinement pressure of \( f_i \), the required CFRP jacket thickness is:

\[ t_j = \frac{f_i D}{2E_j \varepsilon_j} \]  

(8)

Where \( t_j \) is the jacket thickness, \( f_i \) is the confinement pressure, \( D \) is column diameter, \( E_j \) is the jacket modulus of elasticity, and \( \varepsilon_j \) is the dilating strain of 0.0015.

**5.3.3. Upgrading Confinement**

Caltrans (2007) seismic retrofit guidelines are recommended for upgrading confinement in substandard columns. A confinement pressure of 300 psi [2.07 MPa] is required at a radial dilating strain of 0.004 in the plastic hinge regions. The design confinement pressure can be reduced to 150 psi [1.03 MPa] at the same dilating strain outside the plastic hinge. The jacket thickness can be calculated using Equation 8 and the dilating strain of 0.004.

**5.4. CFRP Design Tables**

A set of tables was developed to enable bridge engineers to determine the number of CFRP layers required for seismic repair of the earthquake-damaged bridge columns without any calculations. Having the ratio of longitudinal reinforcement, the column aspect ratio, and the column diameter in addition to the observed damage state, the number of required CFRP layers is determined based on the CFRP design tables. The number of CFRP is listed for plastic hinge zone and non-plastic hinge zone, separately. According to Caltrans (2006) the plastic hinge zone length is defined as follows:

\[ L_{zh} = \frac{3}{8} AR \times D \geq 1.5D \]  

(9)

Where \( AR \) is the aspect ratio and \( D \) is the column diameter. Note that no CFRP jacket is required for columns under damage state 1 (DS-1). The assumed jacket properties are listed under the table.
<table>
<thead>
<tr>
<th>Aspect Ratio (AR)</th>
<th>Inside Plastic Hinge Zone</th>
<th>DS-5</th>
<th>DS-4</th>
<th>DS-3 and DS-2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ρ = 1%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D = 36&quot;</td>
<td>D = 48&quot;</td>
<td>D = 60&quot;</td>
<td>D = 72&quot;</td>
<td></td>
</tr>
<tr>
<td>[914 mm]</td>
<td>[1219 mm]</td>
<td>[1524 mm]</td>
<td>[1829 mm]</td>
<td></td>
</tr>
<tr>
<td>1.5 ≤ AR</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Outside Plastic Hinge Zone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5 ≤ AR</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>2</td>
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<td>2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>3.5 ≤ AR</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>ρ = 2%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D = 36&quot;</td>
<td>D = 48&quot;</td>
<td>D = 60&quot;</td>
<td>D = 72&quot;</td>
<td></td>
</tr>
<tr>
<td>[914 mm]</td>
<td>[1219 mm]</td>
<td>[1524 mm]</td>
<td>[1829 mm]</td>
<td></td>
</tr>
<tr>
<td>AR &lt; 2</td>
<td>4</td>
<td>6</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>5</td>
<td>7</td>
<td>8</td>
</tr>
<tr>
<td>3 ≤ AR</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Outside Plastic Hinge Zone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR &lt; 2.5</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
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<td>2.5</td>
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<td>2</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>4.5 ≤ AR &lt; 6</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6 ≤ AR</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>ρ = 3%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D = 36&quot;</td>
<td>D = 48&quot;</td>
<td>D = 60&quot;</td>
<td>D = 72&quot;</td>
<td></td>
</tr>
<tr>
<td>[914 mm]</td>
<td>[1219 mm]</td>
<td>[1524 mm]</td>
<td>[1829 mm]</td>
<td></td>
</tr>
<tr>
<td>AR &lt; 2.5</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>2.5</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>3 ≤ AR</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>Outside Plastic Hinge Zone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR &lt; 2.5</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
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<tr>
<td>2.5</td>
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<tr>
<td>3 ≤ AR</td>
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<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td><strong>ρ = 4%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D = 36&quot;</td>
<td>D = 48&quot;</td>
<td>D = 60&quot;</td>
<td>D = 72&quot;</td>
<td></td>
</tr>
<tr>
<td>[914 mm]</td>
<td>[1219 mm]</td>
<td>[1524 mm]</td>
<td>[1829 mm]</td>
<td></td>
</tr>
<tr>
<td>AR &lt; 3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>3 ≤ AR</td>
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<td>Outside Plastic Hinge Zone</td>
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</tr>
<tr>
<td>AR &lt; 3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>3 ≤ AR</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
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</tr>
<tr>
<td><strong>ρ = 5%</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inside Plastic Hinge Zone</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5 ≤ AR</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>No Jacket for Outside the Plastic Hinge Zone</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The number of layers is based on concrete strength of 4 ksi [27.6 MPa] and CFRP fabrics with a thickness of 0.04 in [1 mm] per layer, the modulus of elasticity of 10000 ksi [68950 MPa], and an ultimate strain of 0.01.
6. **Important Observations**

1. The spirals in high shear standard columns yielded under DS-4 (visible lateral and/or longitudinal bars) and DS-5 (imminent failure); however, those of low shear standard columns did not yield under any damage states.
2. The transverse steels in sub-standard columns yielded under DS-3 (extensive cracks and spalling but no visible reinforcement). Extensive shear cracks and/or spalling in substandard columns indicated imminent failure.
3. Flexural cracks (DS-1) appeared in low shear standard columns under approximately the effective yield displacement of the column; however, they appeared in high shear standard columns before effective yielding of the columns.
4. Standard columns in the near-field zone experienced the largest drift ratios compared to columns in other categories at all damage states due to the impulsive nature of near-field earthquakes.
5. The entire proposed rapid repair procedure was completed in less than four days for each column including two days of curing for CFRP jacket.
6. The mechanical properties of the CFRP jacket were comparable with the specified properties after 24 hours of accelerated curing followed by 24 hours curing under ambient conditions. Note that specifications call for seven days of curing for CFRP jacket ambient condition.
7. Pressurized epoxy injection of the cracks was the most time-consuming step in the proposed rapid repair procedure.
8. The concrete and spirals outside the plastic hinges maintained their original shear capacity after damage.
9. The strength and the displacement ductility capacity of standard columns were fully restored by the repair.
10. Due to stiffness degradation of the steel and the concrete during the original tests and uninjected micro cracks, the stiffness of the columns was not fully restored by the repair. CFRP jacket substantially reduced shear cracks opening under moderate and large motions.
11. The existing material models did not lead to an accurate estimate of the repaired column stiffness. However, the proposed simple method to modify the steel properties led to a good estimate of the stiffness of the repaired column models.

7. **Important Conclusions**

1. The proposed number of damage states was reasonable in terms of the number distinguishable apparent damage in columns.
2. The proposed rapid repair procedures using CFRP is practical and effective in restoring the shear strength and displacement ductility capacity of earthquake-damaged standard RC columns.
3. Even yielded spirals are effective in providing approximately 50% of their original contribution to the column shear strength in repaired columns. However, yielded spiral contribution to confinement is negligible.
4. In all column models, the concrete and spirals outside the plastic hinges maintained their original shear strength due to a lack of damage.
5. The required confinement in plastic hinges damaged to DS-5 to restore the displacement ductility capacity was consistent with Caltrans provisions for retrofit.
6. Earthquake-damaged substandard columns should be repaired using the current seismic retrofit recommendations as long as there are no fractured bars.
7. The proposed method to model the longitudinal steel in the repaired columns is a simple yet effective method to account for stiffness degradation of the repaired columns.
8. Even though the repair process was done rapidly and was treated as “emergency” repair with implication that it was a temporary measure, it can be treated as a permanent repair as long as the stiffness of repaired columns is sufficient for non-seismic loads.