Rapid repair of RC bridge columns subjected to earthquakes

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ABSTRACT: The middle bent of a large-scale two-span bridge model, which was damaged to the highest repairable level (visible bars, initial buckling in some longitudinal bars, and initial concrete core damage) in the previous tests, was repaired using unidirectional carbon fiber reinforced polymer (CFRP) jacketing and retested to evaluate the repair performance. The concrete was repaired by removing the loose concrete and using a fast set grout. Adjacent cracks were epoxy injected. The columns were then wrapped with CFRP sheets and were cured only for 54 hours (24 hours accelerated, and 30 hours regular curing) before testing. The specifications call for seven day curing but the objective here was to determine the effectiveness of the repair method for emergency repair. The tests showed that the lateral load capacity and the ductility capacity of the bent were fully restored, and the service level stiffness of the bent was restored to 87% of the undamaged bent stiffness.

1 INTRODUCTION

Past effort in the seismic design of concrete bridges has been on detailing of bridges to prevent collapse. During earthquakes, reinforced concrete bridge columns are designed to undergo cracking, spalling, and yielding of steel and provide significant rotational capacity at plastic hinges so that the integrity of the overall structure is maintained. With proper design and construction, this objective can be met. However, the serviceability of the bridge after the earthquake is in question. 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. They should also recommend repair methods for the columns. Any delay in opening of 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.

In this study, a large-scale two-span bridge model, which was damaged to the highest repairable level in the previous tests, was repaired using CFRP wrapping. At this level of the damage, many spirals and longitudinal bars are visible, some of the longitudinal bars are beginning to buckle, and the edge of concrete core is damaged. No bars are ruptured.

2 TWO-SPAN BRIDGE MODEL

The bridge model was a one-quarter scale two-span reinforced concrete bridge supported on three two-column bents, each supported on one of the UNR shake tables. Figure 1 illustrates the general view of the test set up. The bent that was the subject of emergency repair was the middle bent (Fig. 2) because it underwent approximately 8% drift during the previous dynamic shake table tests and experienced the most severe damage. The bent was pushed statically to 10% drift after the dynamic tests to impose more damage in the core without failing the bars. The drift is defined as the ratio of the lateral displacement at the top to the column height. The maximum longitudinal bar strain was 72,674 microstrains (approximately 31 times the yield strain), and the maximum spiral bar strain was 1,436 microstrains (74% of yield). Figure 3 shows the damage at one of the plastic hinges.

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The yield stress for the longitudinal bars and the spirals were 67.94 ksi (468 MPa) and 55.84 ksi (385 MPa), respectively. The concrete compressive strength at the test day was 6.47 ksi (44.6 MPa) on the standard cylinder.


Figure 1. Two-span bridge test set up

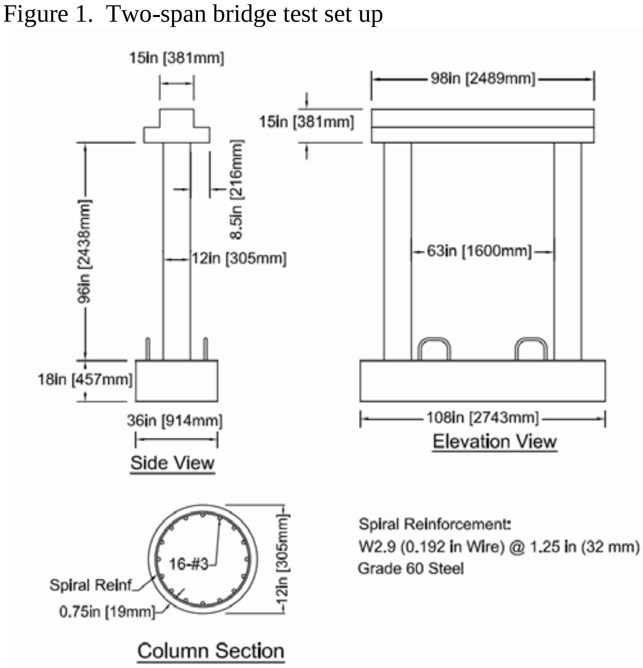

Figure 2. Middle bent (Bent-2) details

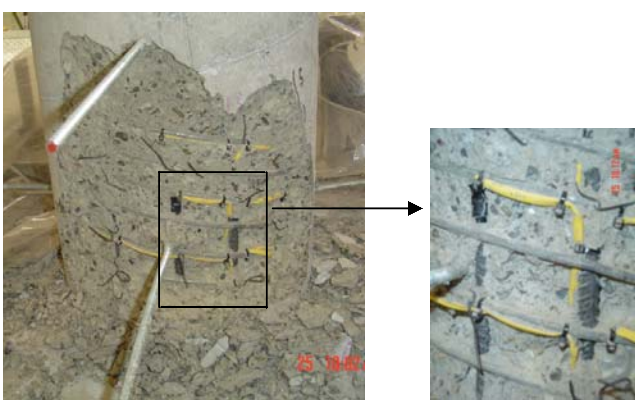
Figure 3. Damage in West column at bottom; West view

3 REPAIR DESIGN

The repair system was designed with the objective of restoring confinement and shear strength of the columns by using unidirectional CFRP jacketing.

3.1 *Restoring confinement*

California Department of Transportation (Caltrans) provisions for RC columns retrofit was used to restoring confinement using FRP jacketing. Based on the provisions, for regions inside a plastic hinge region, without a lap splice, it is necessary to provide a minimum confinement stress of 300 psi (2.07 MPa) at a radial dilating strain of 0.004. For regions outside of the plastic hinge region, the criteria may be reduced to a minimum confining stress of 150 psi (1.03 MPa) at a radial dilating strain of 0.004. The plastic hinge zone is defined as 1.5 times the cross sectional dimension in the direction of bending.

The required jacketing thickness is calculated as follows:

$$t_j = \frac{f_l D}{2\alpha_j E_j \varepsilon_j} \qquad (1)$$

Where t_j is jacketing thickness, f_l is confinement stress, D is column diameter, α_j is reduction factor for CFRP modulus of elasticity, E_j is CFRP modulus of elasticity, and ε_j is dilating strain as defined above.

3.2 *The shear strength restoring*

Priestley (1996) proposed that in calculating the shear resistance contributed by the FRP, the stress in the FRP shall be limited to $0.004E_j$ for a strain limit of 0.004 to avoid degradation in concrete aggregate interlock.

Using Caltrans criteria for seismic shear design for ductile concrete members, the required thickness for the jacketing, t_j, is determined as:

$$t_j = \frac{V_o/\phi - (V_c + V_s)}{\pi/2 \times 0.004 \times E_j \times D} \qquad (2)$$

Where V_o is over strength shear, V_c is the concrete shear capacity, V_s is the shear strength provided by the spirals, and ϕ is 0.85. Other parameters were defined previously.

Since spiral experienced a maximum strain for 74% of yield, and some of cracks were not repairable inside the core, V_c, and V_s were neglected. V_o

was assumed to be associated with the maximum moment achieved in the pre-repair tests.

The material properties are listed in Table 1. The lower limits of specified values were used for repair design. The required CFRP thicknesses to restore confinement were 0.058 in (1.5 mm) and 0.029 in (0.7 mm) inside and outside of the plastic hinge zone, respectively. The required CFRP thickness to restore the shear strength was 0.036 in (0.9 mm) on entire column height. Two layers of CFRP were used over the plastic hinge zones with a length of 1.5D=18 in (457 mm) and one layer elsewhere. The repair design sketch is shown in Figure 4. A jacket system with a thickness of 0.041 in (1.0 mm) per layer was used.

Table 1. Properties of the CFRP/Epoxy jacket system

		Modulus of elasticity (GPa)	Tensile strength (MPa)	Minimum rupture strain (microstrain)
Specified		63.1 ± 1.9	937.7±62.1	13482
Tested at UCSD	1-layer	47.9 ± 2.0	574.5±88.3	9751
	2-layer	56.8 ± 3.6	634.6±93.7	10157
Tested at UNR	2-layer	65.3 ± 8.5	586.4±92.8	6682

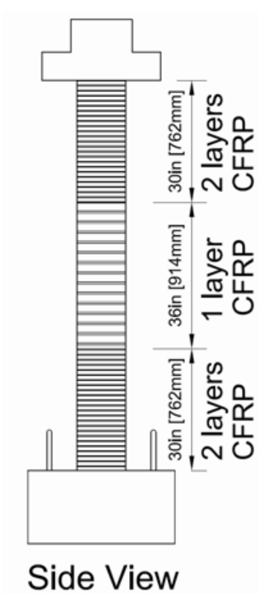

Figure 4. Repair design sketch

4 REPAIR PROCESS

The entire repair process took approximately 30 hours spread over four days, and it involved the following steps:

4.1 *Straightening the columns*

By adjusting the shake tables, the columns were returned to the initial vertical position. Figure 5 shows the middle bent before and after the straightening.

4.2 *Concrete chipping*

In this step, only the loose concrete was removed. Figure 6 shows a hinge before and after the concrete chipping.

4.3 *Pressurized epoxy injection of the cracks*

To inject the epoxy, an inlet was put at one end of a crack and an outlet was put at the other end. Then the surface of the crack was covered by removable glue. Epoxy was injected from the inlet until it bled from the outlet to ensure that the crack was completely filled with epoxy. Figure 7 shows the process.

4.4 *Fast set concrete patching*

For concrete repair, A rapid repair mortar was used. The mortar was low-shrinkage, microsilica-modified, cement-based mortar for structural repair or overlays.

The 1-day specified compressive strength and 28-day modulus of elasticity for the mortar are 4 ksi (27.6 MPa), and 3800 ksi (26200 MPa), respectively. Figure 8 shows the repaired surface.

4.5 *Surface preparation for CFRP wrapping*

Column surfaces were roughed by a grinder only in the plastic hinge zones. A layer of the epoxy was applied to prime the columns surfaces. Finally, a specified type of filler was added to the epoxy to get a thickened epoxy and then applied directly to the columns to smooth out imperfections.

4.6 *CFRP wrapping*

After preparing the surface, CFRP layers were saturated in the epoxy and then wrapped around the columns manually. Figure 9 shows the wrapped columns.

4.7 *Curing*

After wrapping CFRP, the temperature was elevated to a range of $94^\circ F$ ($34°C$) to $100^\circ F$ ($38°C$), and the relative humidity was reduced from 19%-21% to approximate 10% . This was accomplished by hanging plastic sheet from the bridge soffit, using four 1000-watt lamps, directed away from the columns, a resistive heater, and a fan. Figure 10 shows the setup.

These conditions were maintained for about 24 hours and the heaters and the plastic encasement were removed to allow installation of strain gages and LVDT's.

After jacketing, some CFRP samples were made and cured in same conditions as the middle bent of the bridge. Both 1-layer, and 2-layer samples were made and tested at UNR and UCSD at the same time as repaired bent was being tested. The results are shown in the Table 1 and compared with specified properties. The last column shows the minimum rupture strain calculated based on the minimum tensile strength and maximum modulus of elasticity. Specified properties are typically based on test results after one week curing. The results show that the modulus of elasticity in the samples were almost the same as specified one. However the minimum rupture strain was lower, but it was larger than design strain, 4000 microstrain. It is therefore concluded that the curing process was effective for the emergency repair.

Figure 5. Middle bent before (left) and after (right) straightening.

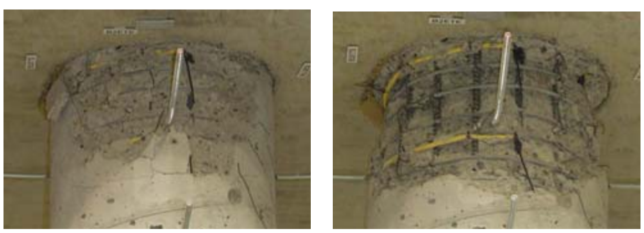

Figure 6. Before (left), and after (right) loose concrete chipping

Figure 7. Epoxy injection

Figure 8. Repaired surface

Figure 9. Wrapped columns

Figure 10. Curing process

5 TEST PROTOCOL

5.1 *Pre-repair loading protocol*

The original 2-span bridge specimen was tested under the near-field motions increasing gradually with simulating the fault rupture. It was assumed that the fault "ruptured" between Bent-2 (middle bent), and Bent-1 (medium height, north bent). Each table shook with an individual ground motion to consider the near-field characteristics of an earthquake.

The most severely damaged bent was Bent-2, with major concrete spalling and exposure of the longitudinal and transverse bars. The core was essentially intact.

It was decided to increase the level of damage to the highest repairable level (core damage; imminent failure) by applying additional static loading. The maximum achieved drift ratio in Bent-2 was 8% and 10% in the dynamic tests and the static tests, respectively.

5.2 *Post-repair loading protocol*

The repaired bridge was tested approximately 54 hours after wrapping. The tests composed of three dynamic tests, eight static tests, and four white noise tests. The dynamic tests were exactly the same as the original tests. After dynamic tests, all the tables were moved to one direction, and the middle table was pushed until the middle bent failed

The maximum achieved drift ratio at post-repair tests in Bent-2 was 13%.

6 REPAIRED BRIDGE PERFORMANCE

For dynamic tests, an envelope was developed based on the peak base shears and the corresponding displacements. The envelope was continued by adding base shears and the corresponding displacements from static tests. The envelopes are illustrated in the

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Figure 11, and 12 for pre-repair and post-repair tests respectively.

The first significant FRP jacket was observed in Test 07. During this test, the FRP jacket ruptured on the west face in the bottom side of the west column (Fig. 13). The rupture was accomplished with a loud noise. After that, there was a drop in the bent base shear. The second FRP breakage was heard in Test 10, but no more rupture was apparent. In Test 11, two more noises were heard due to the rupture of FRP. The FRP ruptured on the west face in the bottom side of the east column (Fig. 14).

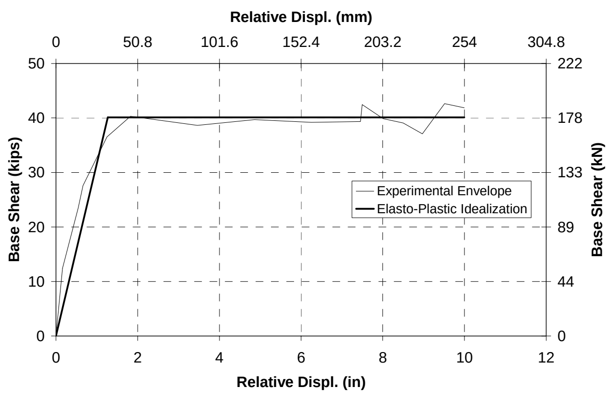

Figure 11. Force-displacement relationship for pre-repair tests

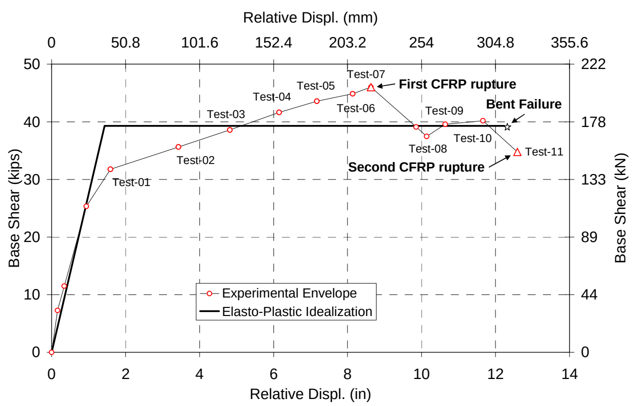

Figure 12. Force-displacement relationship for post-repair tests

Figure 13. Rupture in Test-07

Figure 14. Rupture in Test-11

To allow the comparison of the responses with respect to lateral load capacity, service level stiffness, and the ductility capacity, the measured envelopes were idealized by elasto-plastic curves. The pre-repair envelope was idealized by setting the initial slope to pass through the first yield point and adjusting the plastic portion so that areas above and below the idealized curve were balanced with the original backbone curve. The elastic part for post-repair idealization was obtained by connecting the origin to a point on the envelope curve at which the force was one-half of the peak value for that curve. The yield level was established by equalizing the area between the measured and idealized curves. Failure was assumed to occur when the lateral load had dropped by 20 percent compared to the peak lateral load. The idealized elasto-plastic curves are shown in the Figure 11, and 12 for pre-repair and post-repair tests, respectively. The idealized response values for pre-repair and post-repair tests are listed in the Table 2.

Table 2. Idealized response values for pre-repair and post-repair tests

Plastic base shear (kN)		Effective yield displacement (mm)		Ultimate displacement (mm)	
Pre-repair	Post-repair	Pre-repair	Post-repair	Pre-repair	Post-repair
178.4	174.7	32.2	36.4	253.9	310.9

The ratio of plastic base shear to effective yield displacement is defined as service stiffness, and the ratio of ultimate displacement to effective yield displacement is defined as ductility capacity. These two values are shown in Table 3.

Table 3. The service stiffness and ductility capacity for pre-repair and post-repair tests

Service stiffness (kN/mm)		Ductility capacity	
Pre-repair	Post-repair	Pre-repair	Post-repair
5.54	4.80	7.89	6.87

Table 2 shows that the capacity of the bent was restored completely after repair. The ductility capacity of the bent was also restored. Although the achieved ductility capacity for post-repair tests is greater than the ductility of pre-repair tests, but it must be noted that the calculated ductility for the pre-repair tests is based on maximum displacement at the highest repairable level, and not failure. In addition, 87% of the elastic stiffness of the bent was also restored by the repair.

The maximum drift ratio in the pre-repair tests, and that of post-repair tests at failure were 10.4%, and 12.75%, respectively. It can be concluded that the ultimate drift capacity of the repaired bent was comparable to that of the original bent.

7 CONCLUSIONS

The middle bent of a large scale two-span bridge model which was damaged to the highest repairable level in the previous shake table tests, was repaired using CFRP wrapping and retested to evaluate the repair performance. At this level of the damage, many spirals and longitudinal bars are visible and some of the longitudinal bars are beginning to buckle. Based on the achieved data from testing the repaired model, the following conclusions are made:

- The repair design method, providing minimum confinement pressure for 300 psi (2.07 MPa), and providing plastic shear strength, was effective and appropriate.
- The repair process was practical and may be used for emergency repair of earthquake damage concrete columns.
- The repair restored the strength, ductility capacity, and drift capacity of the model completely, and restored the initial stiffness up to 87% of the original stiffness.
- Although the jacketing system was cured for only 54 hours (24 hours of elevated heat and 30 hours of ambient lab temperature), the jacket system had the modulus of elasticity equal to specified value after one week curing. The minimum ultimate strain was less than specifications, but it was larger than the maximum design strain. Note that the specified curing time for CFRP is seven days.

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