REPAIR OF HIGH SHEAR STANDARD REINFORCED CONCRETE BRIDGE COLUMNS USING CFRP

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Abstract

Two circular, high shear RC columns were designed identically using current bridge seismic codes. They were damaged to near failure using one of the shake tables at the University of Nevada, Reno. The columns were repaired using unidirectional carbon fiber reinforced polymer (CFRP) jacketing and retested to evaluate the repair performance. The loose concrete was removed and the spalled area was repaired using a fast-set non-shrink mortar. The cracks were epoxy injected. Different number of CFRP layers and different repair mortar and application method were used for the columns. The results indicate that the strength and drift capacity of the columns were fully restored. In addition, it was found that spirals maintain approximately 50% of their capacity even when the column damage is severe.

Introduction

Past effort in 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. Rapid and effective repair methods are needed to enable quick opening of the bridge to minimize impact on the community.

As part of this study, two 1/3 scale high shear standard RC bridge columns, which were damaged to the highest repairable level in the previous tests, were 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 bar is ruptured.

Column Models

NHS1 and NHS2, <u>N</u>ew-design <u>H</u>igh <u>S</u>hear, were the two identical double-curvature column models that were studied. The double-curvature configuration allows for the application of a relatively high shear, resulting in extensive shear cracks

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in concrete that induced high demand in the transverse steel, in addition to flexural cracks and plastic hinging. Note that the columns were flexure dominated and expected to be ductile. The latest Caltrans Seismic Design Criteria, SDC version 1.4 (2006), was used to design the column. Scale factor of 1/3 was selected based on the typical cross section dimensions of bridge columns. The scale factor was applied in a way that stresses would not be scaled and real concrete and steel could be used. NHS1 and NHS2 were repaired using fast set non-shrink concrete, epoxy injection, and CFRP wrapping. Different concrete repair methods and jacket layers were applied for NHS1 and NHS2. The former repaired column and the latter one are called NHS1-R and NHS2-R, respectively. The column specifications are listed in **Table 1**. The test setup and column section are shown in **Fig. 1**. The primary test variable was the number of CFRP wraps. The objective was to determine if by reducing the number of wraps and counting on partial contribution of the column spiral to shear would lead to satisfactory performance.

Repair Design

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

Restoring confinement

Because there are no seismic *repair* design guidelines available, seismic *retrofit* guidelines in the California Department of Transportation (Caltrans) provisions for RC columns were used to restore 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 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 length of the plastic hinge zone is defined as 1.5 times the cross sectional dimension in the direction of bending.

The required jacket thickness is calculated as follows:

$$t_j = \frac{f_l D}{2E_j \varepsilon_j} \tag{1}$$

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

Restoring shear strength

Priestley (1996) recommended 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. Combining the recommendation and the 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 spirals, and ϕ is 0.85. Other parameters were defined previously. V_o was assumed to be associated with the maximum moment achieved in the NHS1 and NHS2 tests. Different assumptions were made for inside and outside the plastic hinge zone to calculate V_c and V_s:

Inside the plastic hinge

Since some of the thin cracks are not repairable inside the core, V_C was neglected in the both columns. The spirals for NHS1 experienced a strain greater than 1.75 yield strain. As a result, V_s was assumed to be zero for NHS1-R. Testing NHS1-R on a shake table and calculating the contribution of the spirals to shear showed that spirals resisted approximately 50% of the shear even though they were neglected in design. In NHS2-R this led to a smaller number of CFRP Layers.

Outside the plastic hinge

Since the spirals do not yield outside the plastic hinge, a 100% credit was given to spirals in both columns. Although shear cracks occurred outside the plastic hinge too, the level of damage was much lower than that of the plastic hinge. As a result, 50% credit was also given to the concrete outside the plastic hinges. A jacket system with a thickness of 0.04 in (1.0 mm) per layer was used.

Repair Process

Neglecting the unexpected delays, the entire repair process can be conducted in one day. The repair process is shown in **Fig. 2** for one of the plastic hinges. The repair process consisted of the following steps:

Straightening the column

By adjusting the shake table, the column was returned to the initial vertical position.

Removing loose concrete

The loose concrete was removed by an impact hammer with a chisel point. Mostly, the loose concrete was removed from the compression dominant side of the column.

Concrete repair

Two different types of mortars and mortar placement were used for NHS1-R and NHS2-R. In NHS1-R, a one component, micro silica and latex modified, and non sag repair mortar was used. The specified 3-day compressive strength of the mortar in NHS1-R was 4000 psi (27.6 MPa). In NHSR-2, a rapid repair mortar was used. The mortar was low-shrinkage, microsilica-modified, cement-based mortar for structural repair or overlays. The specified 3-hour and 1-day compressive strengths of the mortar used for NHS2-R were 3000 psi (20.7 MPa) and 4000 psi (27.6 MPa), respectively.

Pressurized epoxy injection of the cracks

The epoxy was injected into shear cracks and flexural cracks on the tension dominant side of the column. 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 with a standard pressure of 40 psi (0.28 MPa) from the inlet until it bled from the outlet to ensure that the crack was completely filled with epoxy.

Surface preparation for CFRP wrapping

Column surface was roughened slightly by a grinder. A layer of epoxy was applied to prime the columns surfaces. After that, a thickened epoxy was applied directly to the columns to smooth out imperfections.

CFRP wrapping

After preparing the surface, the epoxy was applied to CFRP layers using a paint roller. CFRP layers were wrapped around the columns manually.

Curing

The entire curing took less than 48 hours which was composed of first half of accelerated curing, followed by curing under the lab ambient condition. During accelerated curing, the temperature was elevated to $100^\circ F$ to $112^\circ F$, and the relative humidity was reduced to 10% to 15% .

Test Protocol

The Sylmar ground motion record was applied in shake table testing of the columns. The record was applied to the columns with amplitudes increasing gradually. The objective of NHS-1 and NHS-2 testing was to reach to the imminent failure state. In this damage state, the column is approaching failure and damage has begun to penetrate the confined core. No bar rupture is desired in this damage state. To evaluate the repair performance, the repaired columns were subjected to identical increasing motions as original columns with additional motions having higher amplitudes applied to the repaired columns until failure.

Shake Table Tests Results

The columns were tested on one of the shake tables for University of Nevada, Reno. The failure mode for NHS1-R was fracture of two longitudinal bars at the base (**Fig. 3**). In NHS2-R failure was due to the CFRP rupture at the column base on the compression dominant side along with rupture in two longitudinal bars at the base (**Fig. 4**). In both columns, bar ruptures were noted by removing the CFRP jacket.

The cumulative force-displacement hysteresis and back bone curves for NHS1-R and NHS2-R are shown in the upper graphs in **Fig. 5**. In the lower left graph, back bone curves for NHS1 and NHS1-R are plotted. Those of NHS2 and NHS2-R are plotted in the lower right graph.

Strength, stiffness, and deformability are the main characteristics of a structure. To compare the performance of the original columns and repaired ones, the fallowing non-dimensional response indices were developed. These indices can also be used among repaired columns to compare the performance of different repair methods.

Strength Index (STRI)

The lateral strength of a column is defined as the peak measured base shear. The ratio between the lateral strength of the repaired column and the original one is defined as strength index:

$$STRI = \frac{V_R}{V_O} \tag{3}$$

Where, V_R and V_O are the peak base shears for the repaired column and the original one, respectively.

Stiffness Index (STFI)

The serviceability of a repaired structure also needs to be considered. The stiffness of the structure under low amplitude lateral loads is an important parameter for quantification the serviceability. Assuming a point on the push over curve with one-half of the peak base shear as the elastic limit, the chord service stiffness of the column is calculated by dividing the one-half of the peak base shear by the corresponding displacement. Having the chord service stiffness for the original column and the repaired one, the stiffness index is found as follow:

$$STFI = \frac{K_R}{K_O}$$

Where, K_R and K_O are the chord service stiffness for the repaired column and the original one, respectively. It should be noted that the elastic limit of the repaired column is not taken larger than that of the original column.

Deformability Index (DI)

This index is defined as the ratio between the drift capacity of the repaired column (D_R) and that of the original column (D_O). Deformability index is determined as follows:

$$DI = \frac{D_R}{D_O} \tag{4}$$

Since the original columns were not tested to failure, their drift capacity is larger than the maximum measured drift. Past failure test data have shown that for well designed columns under high shear, the ratio of ultimate displacement to displacement at imminent failure is approximately 1.2. As a result, to calculate *DI*, the maximum measured drifts for NHS1 and NHS2 were increased by 20%.

The peak base shear, maximum drift, and service stiffness for the columns are listed in **Table 2**. These parameters were used to calculate the response indices. Although the strength of NHS2 was less than that of NHS1, the repaired columns had equal strengths. NHS2-R had considerably higher service stiffness than that of NHS1-R, however those of NHS1 and NHS2 were almost the same. The table also shows that the maximum drift for both repaired column was nearly the same.

The response indices are plotted in **Fig. 6**. In general, all the response indices for NHS2-R are higher than those of NHS1-R. It implies that the repair procedure in terms of quality and application method of the repair mortar has significant role in the performance of the repaired column. In addition, the number of CFRP layers in NHS1-R was not optimized and giving 50% credit to the spirals for NHS2-R was a reasonable assumption. **Fig. 6** shows that the strength and deformability of both columns were fully restored. Although the service stiffness was not fully restored in both columns, but the stiffness reduction in NHS2-R was 2/3 of that of NHS1-R. The reason is that the higher quality repair mortar and better application method, pouring and vibrating instead of patching, were used in NHS2-R.

Conclusions

Based on the observations and the measured data from the testing of the original and the repaired columns, the following conclusions are made:

- The repair design method was rapid, and effective because it restored the lateral load and drift capacity of the columns.

- The repair process was practical and may be used for rapid emergency repair of earthquake damaged concrete columns.

- Giving 50% credit to the spirals capacity and neglecting the concrete strength inside the plastic hinges is a reasonable assumption in the repair design.

- Giving full credit to the spirals capacity and 50% credit to the concrete strength outside the plastic hinges is a reasonable assumption in the repair design.

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Table 1. Specifications for NHS1 and NHS2

H in (mm)	D in (mm)	Long. Steel Ratio	Trans. Steel Ratio	Aspect Ratio	Axial Load, kips (kN)
80 (2032)	16 (406)	3.08%	1.34%	2.5	100 (444.8)

Table 2. Main responses of the columns

	Peak base shear, Kips (kN)	Maximum drift	Service stiffness, Kips/in (kN/mm)
NHS1	94.1 (418.6)	9% [1.2*7.5%]	73.2 (12.8)
NHS1-R	95.2 (423.6)	13.1%	28.3 (5.0)
NHS2	78.9 (350.9)	7.7% [1.2*6.4%]	71.0 (12.4)
NHS2-R	92.1 (409.7)	13.3%	44.0 (7.70)

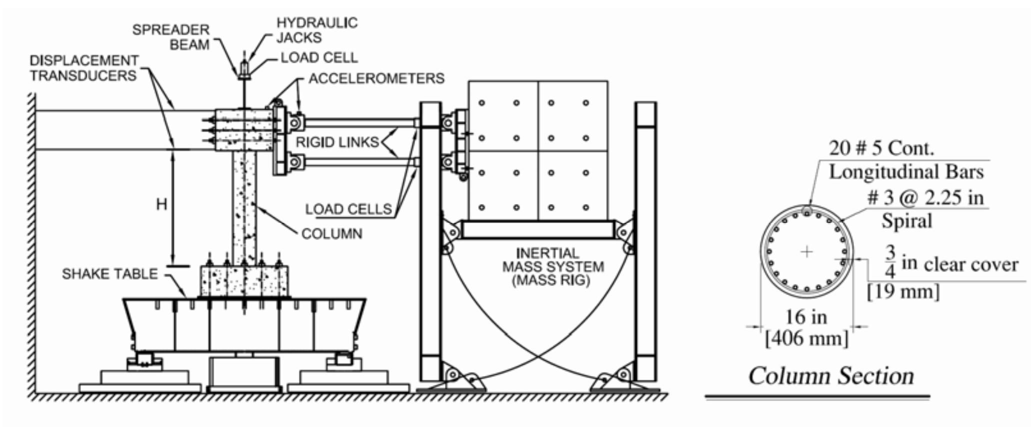

Fig 1. Test setup and section propertis for NHS1 and NHS2

2a) Before repair

2b) Concrete Chipping

2c) Concrete patching (NHS1-R)

2d) Smoothening (NHS-R)

2e) Conc. pouring, vibrating (NHS-R)

2f) After concrete repair

2g) Epoxy injection

2h) Surface preparing

2i) CFRP wrapping

Fig 2. Repair process for NHS1 and NHS2

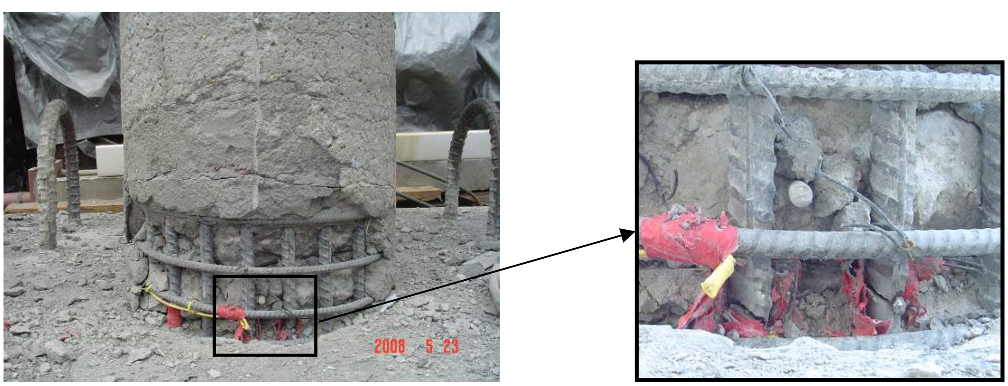

Fig 3. Bar ruptures in NHS1-R. (CFRP was removed after shake table tests.)

(a) **(b)**

Fig 4. Failure in NHS2-R; a) CFRP rupture; b) bar rupture (CFRP was removed after shake table tests.)

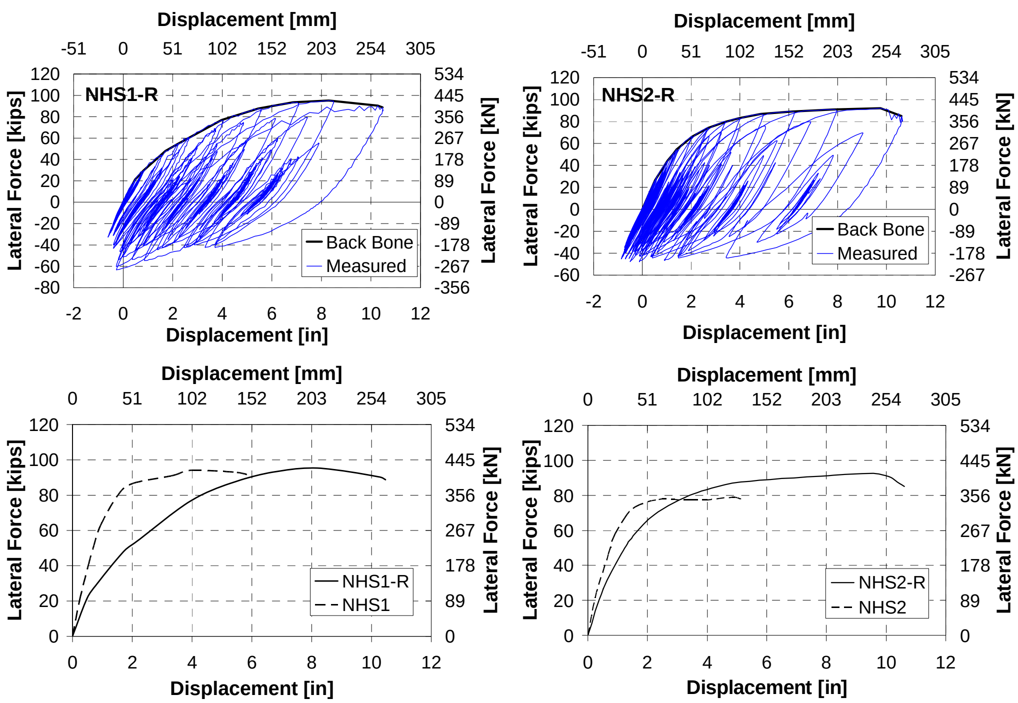

Fig 5. Lateral force-displacement relationships for the columns

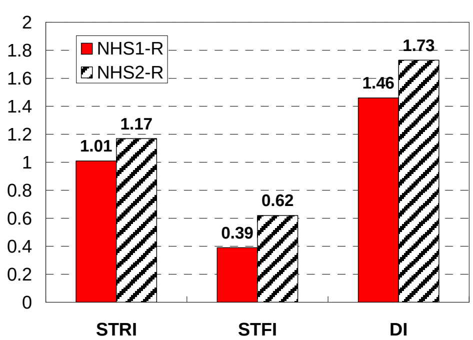

Fig 6. Response indices for the repaired columns