Shake Table and Analytical Investigations of Single Column Bents

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ABSTRACT
Results of shaking table experiments and analyses are summarized which highlight efforts to improve understanding of the behavior of modern bridge structures to severe ground motion shaking, evaluate and improve analytical modeling techniques, assess design guidelines and develop new column details capable of improving post-earthquake operability of bridges. Tests were undertaken on modern circular columns with spiral reinforcement, oblong columns with interlocking spirals, and partially prestressed circular columns. A simple frame from a single column viaduct was also considered. Conventional elastic analyses as well as more refined nonlinear models based on fiber representations of critical sections were considered. Recent work, highlighted in this paper, focuses on a very promising design approach where columns yield like conventional bridge columns, but tend to re-center far more following severe ground shaking.

INTRODUCTION
During strong earthquake ground shaking, reinforced concrete bridge columns are expected to develop high ductility capacity and avoid collapse (California Department of Transportation, 2001). Numerous quasi-static tests have been carried out on bridge columns by imposing predetermined unidirectional or bidirectional displacement histories (for a summary, see Hachem *et al*; 2003, Eberhard 2004). While such tests provide important information regarding the behavior of various details, and valuable information for calibrating analytical models, they do not directly indicate likely behavior under dynamic excitations. For this reason, a series of shaking table tests has been undertaken at UC Berkeley by the Pacific Earthquake Engineering Research Center. Conventional as well as newly developed, self-centering reinforced bridge columns have been tested under unidirectional and multi-directional excitations. Companion analytical studies have been undertaken. Because of space limitations, this paper highlights only a few of these studies.

DYNAMIC BEHAVIOR OF CONVENTIONAL RC BRIDGE COLUMNS
Recent investigations in this study have examined the dynamic response of single column specimens with transverse reinforcement provided by single circular (Hachem 2004) and multiple interlocking spirals (Buckman 2005) under uni-directional and multi-axial shaking (Fig. 1). These studies indicate that the maximum displacement of a column can be larger than predicted solely by the peak displacement suggested by 1D excitations, and that traditional elastic modal combination procedures may not be conservative. However, it appears that current methodologies stipulated in the Seismic Design Criteria (Caltrans 2002) for analysis and detailing are adequate for the moderate and long period bridges, and are perhaps quite conservative in some cases. Columns with interlocking spirals exhibited potential instabilities in the weak axis direction due to geometric nonlinearities. In many cases, it was noted that the bridge columns retained a substantial permanent lateral displacement following severe ground shaking. These averaged about 30% of the peak lateral displacement for the ground motions considered. While a number of commonly used, and more refined, analysis methods were able to predict peak lateral displacements, many of the

currently available procedures were unable to predict accurately the post-peak response, including residual displacements and local damage (such as bar buckling). To help assess the accuracy of current nonlinear analysis methods, a simple two-column frame from a single column viaduct model (Fig. 1c) has also been tested. The response of this specimen is quite complex, and is difficult to analytically predict.

a. Circular columns b. Interlocking spirals c. Single column viaducts

Fig. 1 Recent UC Berkeley shaking table tests of bridge columns

DYNAMIC BEHAVIOR OF SELF-CENTERING RC BRIDGE COLUMNS

As noted above, conventionally designed reinforced concrete bridge columns can achieve large inelastic deformations without significant loss of vertical or lateral load capacity, but may have significant post-earthquake residual displacements, necessitating long-term closure of highways while expensive repairs, or even complete replacement, are carried out. Following the Kobe earthquake more than 100 RC bridge columns were demolished because of residual drift indices exceeding 1.75%. Japanese design criteria (Japan Road Association, 2002) have changed to explicitly require designers to limit permanent drifts to less than 1%.

A recent analytical study (Sakai and Mahin, 2004a & 2004b) proposed a new method to reduce residual displacements by incorporating an unbonded prestressing tendon at the center of a lightly reinforced concrete column. The study shows that (1) incorporating an unbonded prestressing strand at the center of a lightly reinforced concrete column can achieve restoring force characteristics similar to a conventional RC column upon loading, but with much less residual displacement upon unloading (Fig. 2); (2) such self-centering columns perform very well under uni-directional excitations; predicted residual displacements are only about 10% of those of conventionally detailed columns while the peak responses are virtually identical; and (3) unbonding of longitudinal mild reinforcing bars enhances the origin-oriented tendency of the column's hysteresis.

Specimens

Table 1 and Fig. 3 show the specimens tested in this study. Figure 4 shows the test setup. A scaling factor of 4.5 is assumed for the specimens. The column diameter is 406 mm, and the height from the bottom of the column to the center of gravity of the top mass is 2.44 m, resulting in an effective aspect ratio of 6. The design concrete strength is 34.5 MPa. As shown in Fig. 3, the specimens support large concrete blocks that idealize the inertia mass and dead load from the bridge superstructure. The variable α_{total} in Table 1 represents the total axial force acting on the column due to the sum of the dead load plus the prestressing force divided by product of the strength of the concrete at the time of testing and the net cross-sectional area of the column.

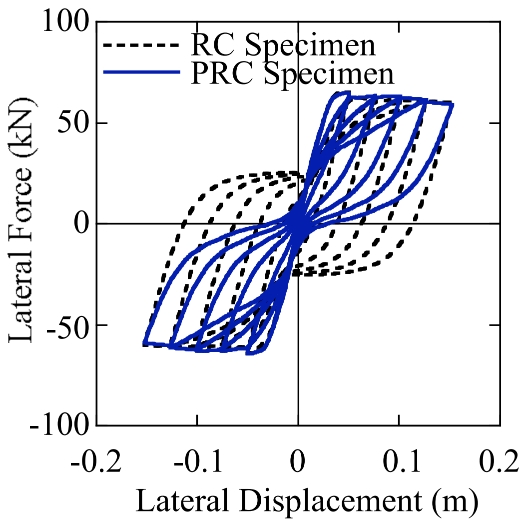
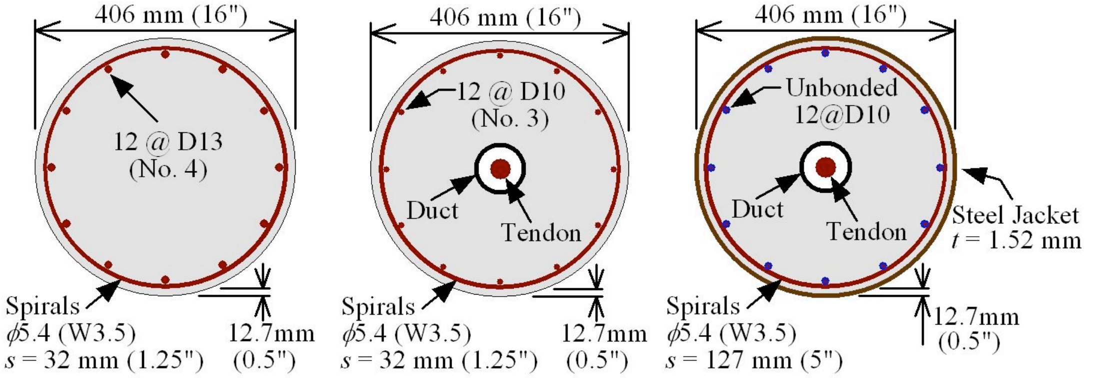


Fig. 2 Comparison of hysteretic loops

Table 1 Specimens

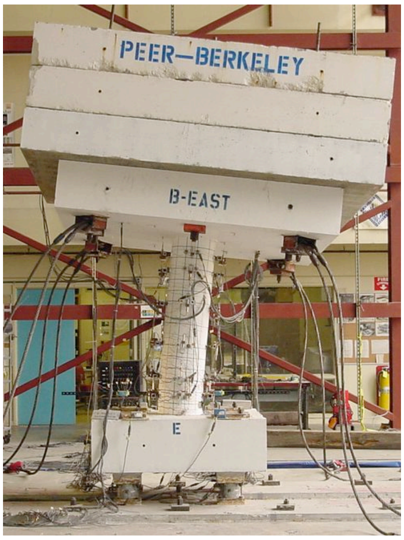
No.	Specimen	Description	f'_{co} (MPa)	P_{ps} (kN)	α_{total} (%)	Tendon Size
1	RC	Reinforced concrete column	41.7	-----	5.4	-----
2	PRC	Partially prestressed RC	41.7	379	12.4	32 mm (1-1/4”)
3	PRC-2	Partially prestressed RC	32.6	220	11.0	36 mm (1-3/8”)
4	PRC-U	PRC-2 w/ unbonded mild bars	32.2	207	10.8	36 mm (1-3/8”)
5	PRC-U2	PRC-U with larger prestressing	32.5	347	14.0	36 mm (1-3/8”)
6	PRC-UJ	PRC-U with steel jacketing	32.1	217	11.1	36 mm (1-3/8”)

The first two specimens were part of an initial pilot investigation. The conventional Specimen RC specimen has a longitudinal reinforcement ratio, ρ_l, of 1.19%, and a volumetric ratio of spiral reinforcement, ρ_s, of 0.76%. Grade 60 bars are used for the mild longitudinal reinforcement, while Grade 80 wire is used for the spirals.



(a) RC specimen (b) PRC specimens (c) PRC specimen with jacketing

Fig. 3 Cross sections considered for self-centering column investigation



(a) Specimen RC (b) Specimen PRC

Fig. 4 Residual displacements of specimens after maximum level test

The design parameters for Specimen PRC were based on results of a series of quasi-static analyses (Sakai and Mahin 2004a). The quantity of longitudinal reinforcement was reduced by half compared to Specimen RC, but a single 32-mm diameter prestressing tendon was placed in an ungrouted conduit at the center of the column to provide nearly the same total longitudinal reinforcement as in specimen RC. The yield and ultimate strength of the tendon are 1024 MPa and 1169 MPa, respectively. With this design, Specimen PRC has a hysteretic envelop during loading very similar to Specimen RC, but with a strongly origin-oriented tendency upon unloading (Fig. 2).

Four more partially prestressed, reinforced concrete specimens were subsequently tested to investigate the effects on seismic behavior of (a) local unbonding of the longitudinal mild reinforcement in the plastic hinge region (PRC-U), (b) the magnitude of the imposed prestressing force (PRC-2U), and (c) adding a steel jacket near the base of the column (PRC-UJ). The basic design of these specimens is similar to that of Specimen PRC, though minor changes were made in the supports for the inertial mass blocks, the tendon area, prestressing force, conduit diameter, concrete strength, and test protocol (Specimen PRC-2). To debond the longitudinal mild reinforcement from the concrete in Specimens PRC-U, PRC-U2 and PRC-UJ, the bars were coated with wax and covered with a plastic sheath for a length equal to 2 times the diameter of the column. The unbonded region begins 152 mm below the footing surface. Specimen PRC-UJ is similar to PRC-U, but a steel jacket (1.52 mm (16 gage) thick) is provided so that the confinement effect of the jacket is similar to that expected in the other columns. The jacket is used as part of the formwork and left in place to provide lateral confinement (only nominal spirals provided).

Ground Motions and Test Sequence

The two horizontal components of a modified version of a record at Los Gatos during the 1989 Loma Prieta, California, earthquake are selected for the test input signals, based on the large residual displacements predicted for the RC specimen by nonlinear dynamic analyses. Both records are scaled using a time scale factor equal to the square root of the length scale factor (= 2.12). Because of the performance characteristics of the earthquake simulator, both components are band pass filtered.

Four intensities of ground motion are imposed in the tests. These levels are denoted herein as the elastic, yield, design and maximum levels. The first two levels are intended to check the instrumentation and data acquisition system, and provide information on the dynamic response of the specimens under excitations representative of moderate earthquakes and aftershocks. The design and maximum level tests investigate nonlinear dynamic response of the specimens. The intensity of the excitations are set to develop a displacement ductility of about 4 during the design level tests, and a displacement ductility of about 9 during the maximum level test (approximately the deformation capacity of the specimen). The intensities of ground shaking were determined based on results of dynamic analyses carried out prior to the first test series. However, these specimens experienced a larger response than predicted for the design and maximum level tests. Thus, the intensities used for the second series of tests were adjusted to better achieve the targeted displacement ductilities. Table 2 summarizes amplitude-scaling factors used for the two test series.

Table 2 Amplitude-scaling factors for ground motion intensities

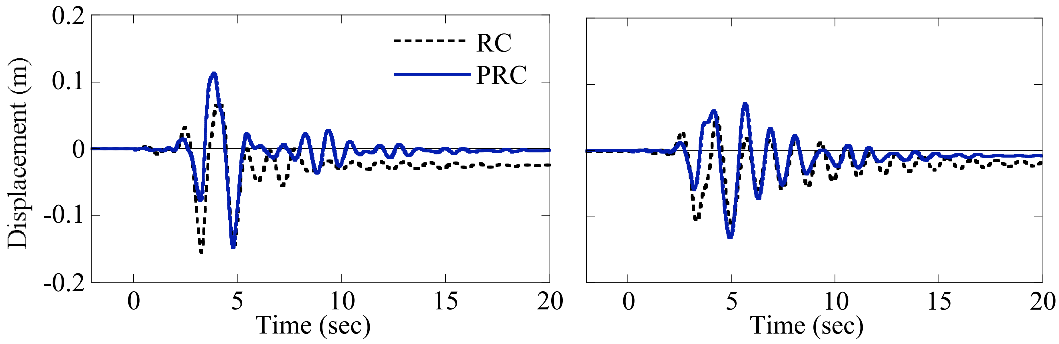
Intensity level	Test Level	Tests in 2004 (RC, PRC)	Tests in 2005 (PRC-2, PRC-U, PRC-U2, PRC-UJ)
1	Elastic	7%	10%
2	Yield	10%	25%
3	Design	70%	50%
4	Maximum	100%	75%

Dynamic Response of Specimens RC and PRC

Figure 5 compares the displacement response at the center of gravity of the top mass subjected to the design level ground motion, and Table 3 shows maximum and residual displacements during the high level tests. The displacements are expressed as distances from the origin in Table 3 while the displacements are shown along each principal direction in Fig. 5. The maximum displacements in the X direction of the specimens are 0.155 m and 0.147 m, respectively, which occurs around 4.8 seconds. About the same time, the specimens reach the maximum distances from the origin, which are 0.187 m and 0.189 m (ductilities of about 7.5). Although both specimens have similar peaks, Specimen RC has a residual displacement of 0.031 m, which is more than 1% drift, whereas Specimen PRC has a residual displacement of only 0.008 m (drift $\leq$ 0.3%). The physical damage in both columns was minor after these tests, consisting of moderate spalling of the cover.

Table 3 Maximum and residual distances of Specimens RC and PRC

Specimen	Design Level (70%)		Maximum Level (100%)	
	Maximum Response	Residual Deformation	Maximum Response	Residual Deformation
RC	0.187 m	0.031 m	0.349 m	0.285 m
PRC	0.189 m	0.008 m	0.323 m	0.107 m



(a) X direction (b) Y direction

Fig. 5 Displacement response at center of gravity of mass blocks (design level test)

Figure 4 shows the residual displacements of the specimens after the maximum level test. The maximum displacement ductility factors attained by Specimens RC and PRC are 14 and 13, respectively. These are very large, exceeding the computed capacities. The residual drift of Specimen RC is more than 10%, while that of Specimen PRC is 3%. Even though Specimen RC suffered such a large residual displacement, no major damage such as crushing of the core concrete, buckling or fracture of the reinforcement was observed. Nonetheless, it was believed that continued testing would be unsafe.

Specimen PRC also did not show severe damage at this stage, and the permanent deformation was much smaller than for Specimen RC. Even though the ductility demand exceeded the theoretical capacity, Specimen PRC was subjected to the design level ground motion again. During the second main pulse, 6 of the 12 longitudinal reinforcing bars fractured, resulting in a significant loss of restoring force, and collapse of the specimen.

Effects of Unbonding Mild Reinforcement and Using Steel Jacketing

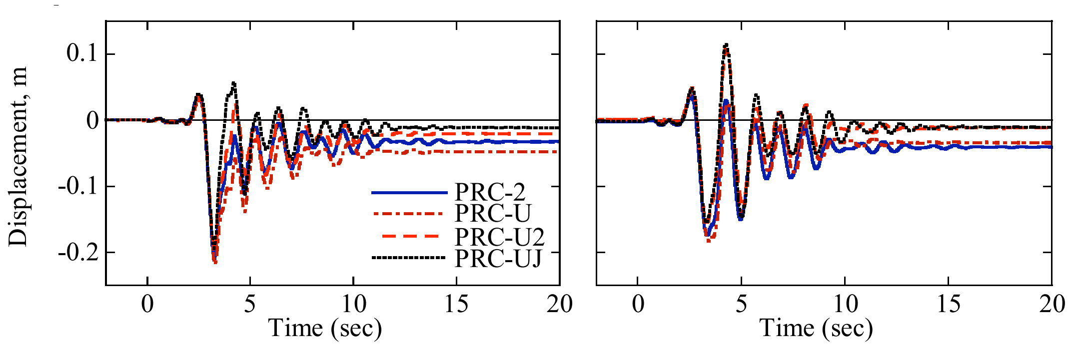
In the second test series, efforts were made to reduce the susceptibility of Specimen PRC to fracture of the longitudinal mild reinforcement and crushing of the confined core. To reduce the maximum strain induced in the bars, the mild reinforcement in the vicinity of the expected plastic hinge were unbonded from the concrete in three of the specimens. In this manner, strains in the bars tend to distribute over the unbonded length rather than localize near large cracks that form during the maximum level events. Buckling of longitudinal bars also accelerates their fracture. Decreasing the pitch of the already closely spaced spiral reinforcement was not a practicable solution. As such, steel jacketing was provided in one specimen. This jacket reduces the need for spiral reinforcement in the column, and is expected to prevent spalling of the concrete cover, thereby obviating the need for, or further reducing the cost of, post-earthquake repair. Because excessive compression forces in the confined concrete can also trigger failures, one test is carried out considering a larger prestressing force.

Table 4 summarizes the maximum and residual displacements at the center of gravity of the top mass block for the second set of specimens. The values are shown as distances from the origin. These specimens all exhibit similar response during the first design level excitation (reaching a displacement ductility of about 5). All of these specimens demonstrate an impressive ability to re-center. Residual displacements for all specimens are smaller than 10% of the yield displacement (a drift of about 0.1%). Damage consists of moderate spalling of the concrete cover, except for the steel jacketed column for which only very minor buckling of the jacket was observed at one side of the column.

Table 4 Maximum and residual distances of PRC specimens

Specimen	Design Level (50%)		Maximum Level (75%)	
	Maximum Response	Residual Deformation	Maximum Response	Residual Deformation
PRC-2	0.117 m	0.002 m	0.269 m	0.052 m
PRC-U	0.124 m	0.002 m	0.278 m	0.058 m
PRC-U2	0.119 m	0.001 m	0.251 m	0.023 m
PRC-UJ	0.123 m	0.001 m	0.245 m	0.015 m

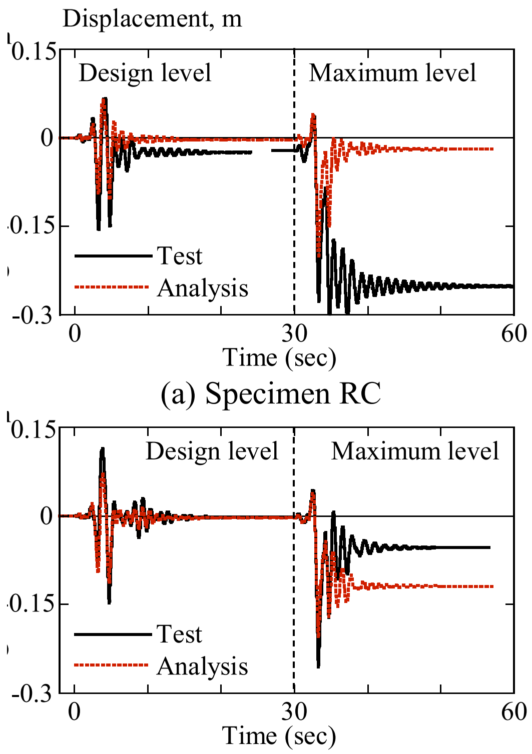
By increasing ground motion intensity by 150% to the maximum level, differences in behavior among the specimens becomes more notable. Figure 10 compares the displacement response at the center of gravity of the top mass of the specimens in the second set subjected to the maximum level shaking. The maximum responses are reached at around 3.3 seconds during the first main pulse in both directions, and are all within 10% of one another. Specimen PRC-U has the largest response, while specimen PRC-UJ has the smallest. The maximum displacements correspond to a displacement ductility of about 10. The residual displacements increase for this severe excitation, but are all less than 0.06 m (< 2.5% drift) and some are much smaller (< 0.6% drift).



(a) X direction (b) Y direction

Fig. 6 Dynamic response of PRC specimens during maximum level test

By using unbonded mild reinforcement, the maximum and residual displacements of PRC-U increase compared to PRC-2 due to smaller flexural strength and even a small negative post-yield stiffness; however by increasing the prestressing force in Specimen PRC-U2, the residual displacement is only 45% of that for Specimen PRC-2. The maximum tensile strains in the reinforcement (measured 0.1 m above the top of the footing) are generally lower for Specimens PRC-U and PRC-U2 than PRC-2, but the maximum width of the cracks near the bottom of the column are larger in these cases. Importantly, the maximum level excitation results in increased spalling and buckling of the longitudinal reinforcement in Specimens PRC-2, PRC-U and PRC-U2. Compared to Specimens PRC-2 and PRC-U, Specimen PRC-U2 (with the higher prestressing force) shows smaller crack opening, more concrete



(b) Specimen PRC

Fig. 7 Analytical simulation showing displacement in X-direction

crushing, and more bar buckling. For Specimen PRC-U, three bars buckled, whereas half of the reinforcement (6 bars) buckled for specimen PRC-U2. When a steel jacket is provided, the peak displacement decreases from 0.278 m (PRC-U) to 0.245 m (PRC-UJ). Similarly, the residual displacement of Specimen PRC-UJ is only 0.015 m (0.6% drift), less than a quarter of that for PRC-U.

The improved behavior of Specimen PRC-UJ at this stage relative to the specimens without steel jackets is believed to be associated with the absence of spalling and, especially, bar buckling. On the other hand, the peak crack opening at the bottom of the jacket is larger than that for any of the other specimens tested. In addition, moderate "elephant foot" buckling is observed intermittently along the bottom of the steel jacket. To mitigate such buckling, a larger gap than provided in the test specimen between the top of the footing and the bottom of the jacket is recommended (as commonly done in California bridge design practice).

ANALYTICAL SIMULATION

Nonlinear dynamic analyses were performed to simulate the behavior of the test specimens. Properly formulated elastic dynamic analyses were able to provide reasonable estimates of peak lateral displacements under 1D excitations. However, modal combination rules underestimated the reinforcing effect of 2D near-fault excitations. Elastic analyses failed to provide any information about residual displacements. As such, a number of nonlinear models were considered. Because of the bidirectional excitation imposed in the tests, the best correlation of global response was achieved when hysteretic behavior is idealized using fiber elements. However, such analyses need further refinement to predict residual displacements and local strains with confidence.

For the partially reinforced concrete columns described above, the reinforced concrete sections were represented by fiber elements. The unbonded tendons and mild reinforcement were idealized with spring elements. Details of the analytical models and assumptions can be found in the report by the authors (Sakai and Mahin, 2004a). Rayleigh viscous damping is typically assumed in the analyses. Measured accelerations at the footing during the tests are used as input for the analyses. Figure 7 compares displacement time histories for the tests and analyses during the design and maximum level tests of Specimens RC and PRC. The analyses for Specimen RC predict 20-30% smaller maximum response, and 80-90% smaller residual displacements. Those for Specimen PRC provide better agreements for the maximum response; however, the computed residual displacements are more than twice the observed test results. Work is continuing to improve the accuracy of predictions of column response, especially residual displacements.

CONCLUSIONS

A broad range of reinforced concrete bridge columns has been the subject of an integrated series of analytical and shaking table investigations. Circular, spirally reinforced concrete columns having proportions, details and gravity loads representative of common California practice exhibit high ductility even under intense near fault ground motions. The presence of moderate damage, such as concrete spalling, and buckling, or even fracture, of longitudinal reinforcement appears to not significantly deteriorate the ability of the bridges to withstand further earthquake motion, provided residual displacements are not large. The residual displacements present in many of the RC columns tested were significant enough to limit traffic flow following earthquakes at or above the design level. Unfortunately, estimates of residual displacement made using current analytical models were found to be inadequate. Additional research is warranted regarding oblong columns with interlocking spiral reinforcement, particularly with regard to frame behavior and their susceptibility to geometric instability.

To reduce residual displacements following strong earthquake ground shaking a series of partially prestressed concrete columns with unbonded post-tensioning tendons were studied. This approach results in columns with approximately the same stiffness and strength as conventional columns, but residual displacements following strong shaking are generally reduced by 70-80%. Peak displacements of the PRC columns are typically within 10% of those for a conventional column. Of the detailing variations studied, it was found that local unbonding of the mild reinforcement in the plastic hinge region can increase fatigue life by reducing the peak strains developed, and steel jacketing combined with locally unbonding of the mild reinforcement can result in columns that attain displacement ductilities near their predicted displacement capacities, but that have little or no visually apparent physical damage and that have residual displacements consistent with continued operation of the bridge following the earthquake. Additional research is needed to refine and confirm design details, especially for actual detailing that would be employed in the field, and for bridge systems incorporating these systems.

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