Repairable precast concrete bridge columns for seismic events

Mostafa Tazarv and Kallan Hart

- The main goal of the present study was to develop reinforced concrete bridge columns that are fully precast, low damage, and repairable through component replacement after being subjected to earthquake loads.

- To achieve the project objectives, 20 repairable precast concrete alternatives were developed and ranked, and the top three candidates were designed at 50% scale, constructed, and tested.

- The repair of the precast ultra-high-performance concrete columns with exposed tendons was easy, simple, and quick due to insignificant damage and minimal residual displacements.

Bridges designed with current seismic codes exhibit large displacement capacities, and bridge collapse is prevented. However, damage of ductile members is allowed at displacements associated with this performance level. In multispan bridges excited by ground shaking, reinforced concrete columns are usually the target ductile members in which concrete cover, core, and reinforcement may be damaged, and the column may not return to its original position. Minor damage is usually repaired, but excess damage, such as core crushing, bar buckling, or bar fracture, is hard to repair and usually results in replacing the column or bridge. According to the Federal Highway Administration (FHWA),[1] 44.52% of U.S. bridges are in good condition, 48.56% are in fair condition, and 6.92% are in poor condition. The fair-rated bridges often need repair, and poor bridges need extensive repair or replacement. Furthermore, within the next 50 years, many bridges located in the 16 seismic-prone states will experience large earthquakes that may cause significant damage.[2] Induced seismic activity in formerly nonseismic states, such as Oklahoma, may also damage bridges with nonseismic detailing. Advanced materials and new detailing can be used to reduce the damage of reinforced concrete bridge columns,[3] thus minimizing the need for repair or replacement after an event. No- to low-damage detailing may be attractive to bridge owners in seismic regions due to the reduced cost associated with the repair or replacement of columns and bridges.

Advanced materials, such as ultra-high-performance concrete (UHPC) and engineered cementitious composites (ECC), may be used in place of conventional concrete to provide some tensile strength and to reduce cracking and spalling of concrete.[4–6] Incorporating different reinforcement, such as stainless steel[7] and microcomposite multistructural formable steel may increase the strength and overall stiffness of reinforced concrete columns while reducing reinforcement corrosion.[8] The use of shape memory alloy (SMA) bars may eliminate residual displacements by providing recentering.[9] Furthermore, fiber-reinforced polymer (FRP) wraps can reduce concrete damage by providing significant confinement.[10]

A few accelerated bridge construction (ABC) details have been proof tested in laboratories and found suitable for bridge columns located in high-seismic regions: pocket connections, mechanical bar splice connections, and grouted duct connections. In a pocket connection, also known as socket connection, a fully precast concrete column can be secured in a large void in the column adjoining member and then be grouted.[11] A type of mechanical bar splice (for example, grouted) is used in mechanically spliced columns.[12] In a grouted duct connection, column longitudinal bars are extended out of the column, are placed in individual ducts, and then are grouted.[13] Columns may be allowed to rock to reduce residual displacements; however, they are axially prestressed or post-tensioned to their adjoining members for enhanced stability. Such columns are usually referred to as *simple rocking* in literature. Simple rocking columns exhibit low energy dissipation and large deformations. When another type of reinforcement is used in addition to the column tensioning reinforcement to increase the energy dissipation and to reduce the displacement demands, the combined system is called a hybrid rocking column.[14–16]

It is feasible to combine new detailing, such as that developed for ABC with advanced materials, to expedite construction, provide additional ductility to reinforced concrete columns, and reduce column damage. For example, a precast concrete column with a rubber bearing that replaces concrete in the plastic hinge region exhibits no damage under large earthquakes.[17]

The main objective of this study was to develop new details for reinforced concrete bridge columns that are fully precast and repairable through component replacement. To achieve this goal, advanced materials and ABC detailing were combined to develop 20 feasible repairable alternatives. All repairable precast concrete column alternatives were evaluated to select the best details for experimental investigation. Of the top candidates, three repairable precast concrete columns were selected, constructed, and tested; and the results were compared with a reference cast-in-place concrete column. The proposed repairable precast concrete detailing alternatives, their ranking, and a summary of the experimental program and findings are presented. Complete discussion can be found in Hart and Tazarv.[18]

Development of repairable precast concrete bridge column alternatives

Modern reinforced concrete bridge columns can usually be repaired at design-level earthquakes. Nevertheless, postevent repair is a passive remedy done based on the type and extent of damage. An alternative repair strategy might be to design reinforced concrete columns to be repairable through component replacement, analogous to a car repair. Hybrid rocking columns with external energy dissipators fit into this repair category. Despite decades of research on hybrid rocking bridge columns, this system has never been used in the United States[3] and only two bridges in the world have been built with this technology.[16] Durability of post-tensioning tendons over 75 years of service life, limited access to internal tendons for inspections, dependence of bridge stability on tendon performance, and somewhat complex design procedures might be the reasons for the limited field application of hybrid rocking bridge columns.

Inspired by rocking detailing and a pilot study on repairable precast concrete moment-resisting buildings,[19] the present study was to investigate the feasibility of a new reinforced concrete bridge column type, referred to in this paper as “repairable,” which has reinforcement that is exposed, accessible, and replaceable using detachable mechanical bar splices (or couplers). In the proposed repairable columns, the exposed replaceable reinforcement is allowed to yield and fracture; however, other column components must exhibit minimal damage (at the reinforcement fracture) for continuous functionality. This is achieved using a capacity-protected design in which the fuse (the exposed reinforcement) is the weak link in the system, and other components have a higher (for example, 25%) strength. The design of repairable columns is further discussed in the section “Design and Construction of Test Specimens.” In a repairable column, the repair is done by replacing the exposed reinforcement (fuses) and should be repeatable.

Following the aforementioned definition and requirements, 20 feasible detailing alternatives were developed for repairable columns (**Fig. 1**). The main components of the proposed repairable columns are a fully precast concrete column to promote ABC, exposed reinforcement, and detachable mechanical couplers to allow reinforcement replacement. The column ends have sections that are smaller in diameter (or side dimension) to accommodate the exposed reinforcement and couplers. This reduced section of the repairable columns is referred to as a *neck region* in this study. In some detailing with a dry connection (for example, ALT-1 in which the precast concrete column seats on the footing), a shear pin is recommended to transfer shear forces. A shear pin is a male-to-female connection in which a steel pipe is secured in a matching socket (or cup). Other optional components of repairable columns are the internal tendons (for example, ALT-17) and high-performance materials (for example, UHPC, engineered cementitious composites, and rubber bearing replacing concrete) to enhance system redundancy—and thus stability—and to improve the seismic performance of the columns, respectively.

If a type of steel bar is used in these details as the replaceable fuse, it must be restrained against buckling to avoid low-cycle fatigue. Buckling-restrained reinforcement (BRR) is a bar that is confined in a steel tube to minimize buckling. BRR can

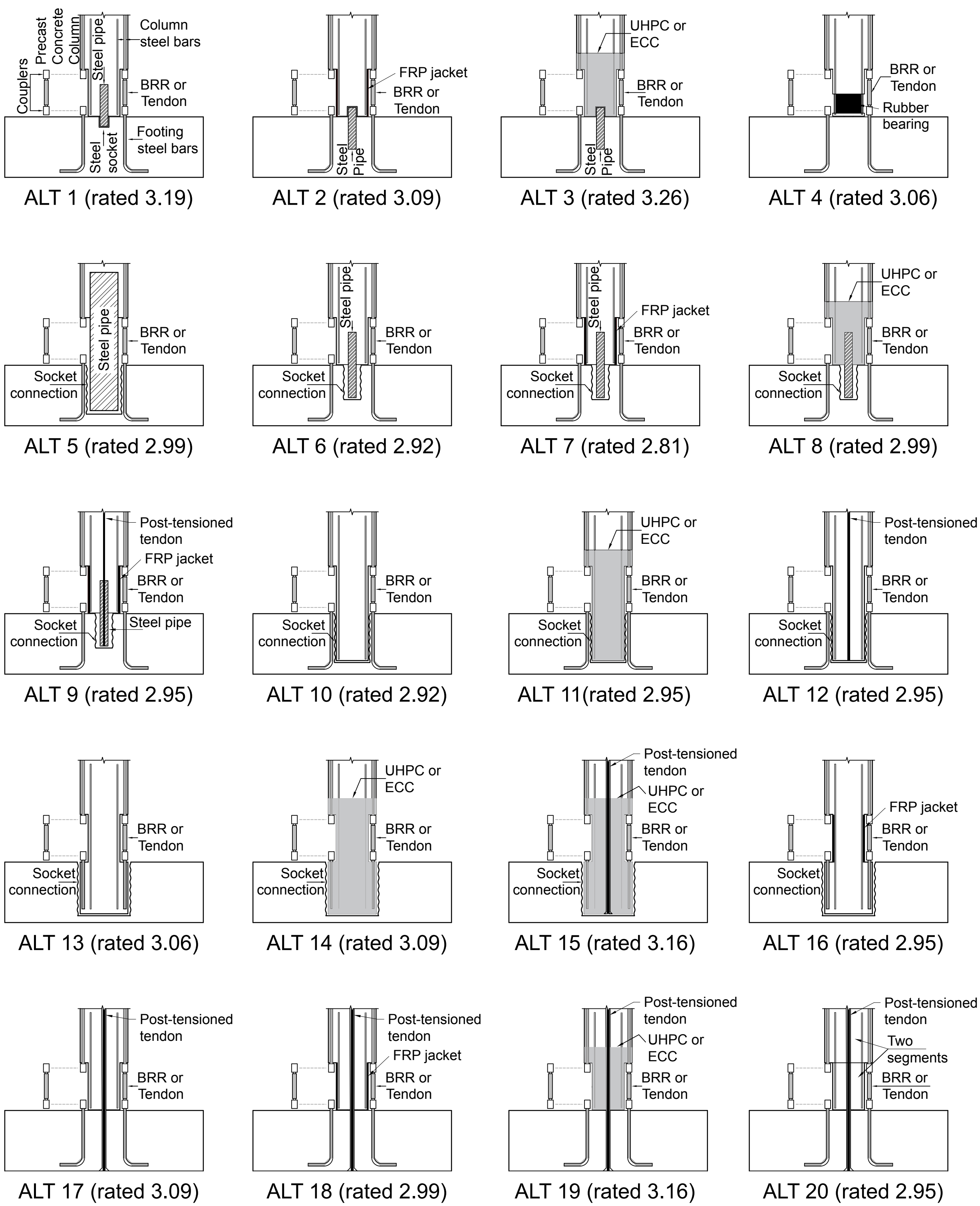


Figure 1. Twenty detailing alternatives and ratings for repairable precast concrete bridge columns. Note: BRR = buckling-restrained reinforcement; ECC = engineered cementitious composites; FRP = fiber-reinforced polymer; UHPC = ultra-high-performance concrete.

be formed with different types of bars, including black steel, stainless steel, SMA, or microcomposite multistructural formable steel. An alternative to BRR is a fuse with tension-only behavior, such as tendons (either steel, SMA, or FRP). A tension-only fuse does not fail by low-cycle fatigue.

All proposed repairable columns are expected to be moment resisting because the tensile-compressive resistance of BRR fuses (at the loading faces of the column) or the interaction between tension-only fuses (at the tensile face of the column) and the bearing of the column concrete against footing (at the compressive face of the column) forms a tension-compression mechanism in the section that resists moments. The shear pin connection (for example, ALT-1), the shear capacity of the column concrete at the column-footing interface (for example, ALT-10), or the shear capacity of the concrete-filled steel tube (for example, ALT-5) resists the column plastic shear forces.

One may assume that repairable columns are the same as simple or hybrid rocking columns. Despite similarities, repairable columns are neither simple nor hybrid rocking columns because internal prestressing or post-tensioning tendons or bars are not required in the proposed details (for example, 13 detailing alternatives shown in Fig. 1, such as ALT-1, ALT-5, or ALT-16). Repairable columns include rocking columns with a broader definition of "any bridge column with replaceable components." As discussed, the external fuses of the 20 alternatives (Fig. 1) are the replaceable components in this study.

NCHRP report 864[3] offers a novel bridge column evaluation method that includes 13 parameters: three parameters related to seismic performance, including plastic hinge damage (at four levels: none, low, moderate, and severe), displacement capacity (low, normal, or high), and residual displacement (low, moderate, or high); four design considerations, including availability of proof testing, availability of analytical tools, availability of design guideline, and past field applications; and six construction considerations, including initial cost, material limitations, ease of construction, inspectability, maintenance, postearthquake repair need, and bridge system performance. In this rating method, each parameter has a range of zero to unity (for example, any column exhibiting a displacement ductility capacity of 3 or less has a score of zero, between 3 and 5 has a score of 0.5, and higher than 5 has a score of 1.0) and is weighted (from 0.25 to 1.0; for example, the weight for the displacement capacity is 1.0, the highest). The weighted scores for all 13 parameters are summed, then the result is converted to a five-star rating for ease of comparison. This quantitative rating system allows uniform comparison of different detailing alternatives for bridge columns at either development or deployment stage. Saiidi et al.[3] provides more discussion on this rating system for novel bridge columns.

This rating system was used to assess the 20 repairable bridge columns and to determine the top candidates for testing. In this project, which was at the development phase, all parameters of the rating system were scored based on existing information on the column constitutive materials without any specific structural analysis. Following this evaluation system (the result of the five-star rating is shown in Fig. 1), the top four columns were identified:

- The best was ALT-3, a precast concrete column with UHPC or ECC in the plastic hinge region and a shear pipe inside footing for a better durability; its total weighted score was 3.26 out of 5.0.

Table 1. Bridge column test matrix

Specimen name	Replaceable reinforcement (fuse) type	Connection type	Column longitudinal reinforcement	Column concrete	Additional remarks
CIP	n/a	Monolithic	Ten no. 8 black steel	Conventional concrete	Cast-in-place vertical construction
RPH-PC	Ten no. 8 stainless steel bars confined in grouted tubes (SS BRR)	Dry with a pipe pin, pipe in the column	Ten no. 10 stainless steel	Self-consolidating concrete	Precast concrete column seating on footing, column tested twice
RPH-PF	Fourteen 0.6 in. diameter, seven-wire steel tendons	Dry with inverted pipe pin	Fourteen no. 10 black steel	Ultra-high-performance concrete	Precast concrete column seating on footing, column tested twice
RPH-NP	Fourteen 0.6 in. diameter, seven-wire steel tendons	Socket with no pin	Fourteen no. 10 black steel	Ultra-high-performance concrete	Hybrid rocking precast concrete column connected to footing via a socket connection, column tested twice

Note: BRR = buckling-restrained reinforcement; CIP = cast-in-place column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing; SS = stainless steel. No. 8 = 25M; no. 10 = 32M. 1 in. = 25.4 mm.

- Second best was ALT-1, which had the same detailing as ALT-3 but with conventional or self-consolidating concrete and steel pipe inside the column; its score was 3.19.

- Tied for third were ALT-15 and ALT-19, with a score of 3.16.

Of these four columns, ALT-1, ALT-3, and ALT-15 were selected for further investigation. Although ALT-19 rates as highly as ALT-15, the post-tensioning of ALT-19 must be performed on-site while the post-tensioning of ALT-15 can be performed at a precasting plant, offering better constructibility and quality. Therefore, ALT-19 was not included in the experimental study. Hart and Tazarv[18] provides in-depth discussions on the evaluation performed for these 20 alternatives.

Experimental program

Test matrix

Table 1 presents the column test matrix, and **Fig. 2** shows the column details selected for testing. The four test specimens can be identified using a naming system including the column and connection type. The reference cast-in-place concrete column is identified with CIP. The name for the repairable columns starts with RPH, which stands for "repairable precast

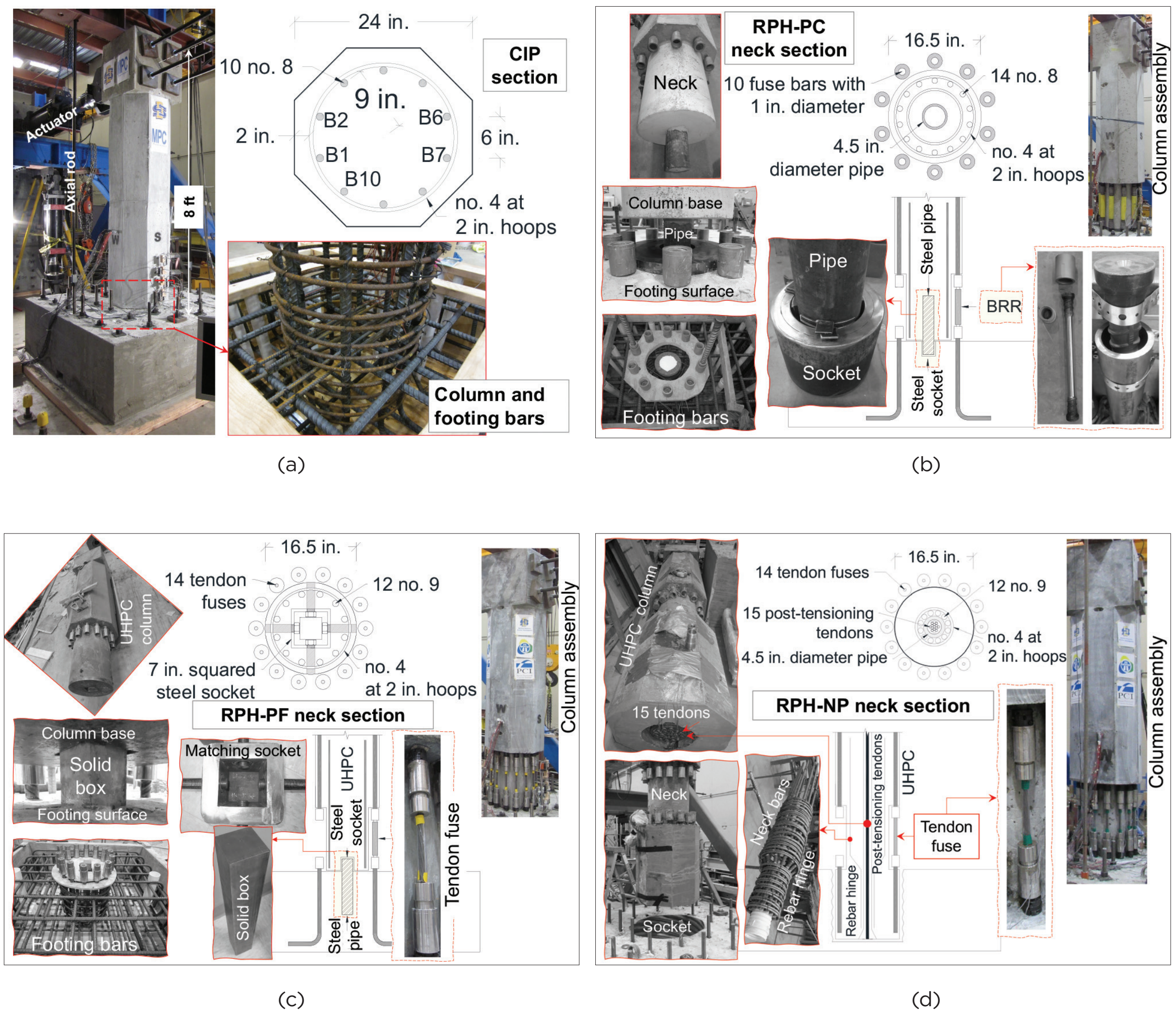


Figure 2. Detailing of half-scale bridge column test specimens. Note: BRR = buckling-restrained reinforcement; CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing; UHPC = ultra-high-performance concrete. No. 4 = 13M; no. 8 = 25M; no. 9 = 29M; 1 in. = 25.4 mm.

concrete column with headed longitudinal bars/couplers," followed by the connection type PC for the column with a pipe-pin connection at the column base with the pipe embedded in the column, PF for a pipe-pin connection with the pipe embedded in the footing, and NP for the column with no pin connection but instead using a socket connection. Later, a third term is added to each column, R, which indicates that the column was repaired by fuse replacement and retested to prove the proposed repair-by-replacement concept. The RPH-PC detailing is based on ALT-1 (Fig. 1), PPH-PF is based on ALT-3, and RPH-NP is based on ALT-15.

Design and construction of test specimens

The geometry of the prototype cast-in-place concrete column was determined based on the results of a recent study[12] on mechanically spliced precast concrete bridge columns, which found that coupler effects were more noticeable in columns with low aspect ratios (the ratio of the column height to the column diameter), low axial loads, and high displacement capacities. The coupler effect was the highest for a column with an aspect ratio of 4, an axial load ratio (the ratio of the column axial load to the product of concrete strength and the column cross-sectional area) of 5%, and a displacement ductility capacity of 7. Although circular reinforced concrete columns are more common and have the most desirable seismic performance due to their uniform confinement, an octagonal cross section with a circular bar arrangement was selected to allow for horizontal placement at precasting plants (for precast concrete columns). In summary, the prototype octagonal cast-in-place concrete column with an aspect ratio of 4 and an axial load ratio of 5% was designed following *AASHTO Guide Specifications for LRFD Seismic Bridge Design*[20] to exhibit a displacement ductility capacity of 7.

Due to setup limitations, a 50%-scale cast-in-place concrete model was selected for testing. The scaling was conducted following the methods discussed in Krawinkler and Moncarz.[21] The scaled CIP model had an octagonal cross section with a medium diagonal of 24 in. (610 mm) and a height of 8 ft (2.44 m) from the top of the footing to the centerline of the applied lateral load, resulting in an aspect ratio of 4.0 (Fig. 2). The cast-in-place concrete column model incorporated 10 no. 8 (25M) longitudinal bars with no. 4 (13M) transverse hoops spaced at 2 in. (50 mm) on center. The resulting longitudinal reinforcement ratio and transverse volumetric steel ratio were 1.66% and 2.0%, respectively. The CIP design concrete compressive strength was 6000 psi (41 MPa) and ASTM A706 bars were considered for all reinforcement. The CIP model design axial load ratio was 5%, equivalent to an axial load of 155 kip (689 kN) for the test specimen. The confinement of the scaled column was also adjusted to achieve a minimum displacement ductility capacity of 7.

A similar cross section to that of the CIP column was used in RPH-PC (Fig. 2) but a pipe-pin connection was devised between the precast concrete column and the footing to allow large rotations at the column base while resisting plastic shear forces. In the neck region, a circular cross section with a reduced diameter of 16.5 in. (419 mm) was used to accommodate exposed reinforcement and couplers. Instead of conventional black steel, stainless steel was used as the longitudinal reinforcement of RPH-PC to minimize the durability issues of exposed components, if such columns were to be used in real applications. Dowel bars matching the number of the replaceable bars (or fuses) were extended out of the footing and the octagonal column to be later connected to the replaceable bars at the time of the column assembly. The headed couplers used in this precast concrete column are detachable, reusable, and low profile. All longitudinal bars and fuses were headed. The female portion of the coupler (the larger piece) was placed on the column and footing dowels and the male portion of the coupler was used on the replaceable bars. Longitudinal bars in the RPH-PC column and footing were oversized to no. 10 (32M) compared with the fuses, which were no. 8 (25M) bars. This was done to ensure that the bar yielding occurs within the fuses but not in the dowels. No. 8 stainless steel bars used in the fuses were machined down from no. 10 bars to match the CIP longitudinal reinforcement area. In the initial testing (fuses colored yellow in Fig. 2), the dog-bone length was 10.25 in. (260.3 mm), while the reduced diameter length was 5.125 in. (130.2 mm) in the second testing, in which the column was repaired (fuses colored in green, shown later). Different fuse lengths of stainless steel BRR were to investigate their effects on the overall column behavior. To prevent buckling of BRR, the core stainless steel bar of BRR was placed inside a steel tube (Fig. 2) then filled with a high-strength nonshrink grout. The design of BRR was based on a previous experimental study.[22] A clear cover of 1 in. (25 mm) was used in the RPH-PC column.

The neck section of RPH-PC was designed assuming that a secondary moment occurs in the opposite direction of the main column moment due to the pin shear forces. The shear force in the pipe-pin connection generates a moment at the top end of the neck region, which is usually larger than the column moment at the same location. This was learned in a previous experimental study on repairable precast concrete beam–column connections.[19] To design the neck region, a moment-curvature analysis was performed on the neck with external fuses placed outside of its cross section, simulating the replaceable fuse properties and location. The resulting maximum moment was divided by the column length to determine the corresponding lateral force at the column base, which was then multiplied by the length of the neck, 24 in. (610 mm), to determine the neck maximum moment. This moment was further increased by 20%, including the material overstrength, and was used to design the neck. For RPH-PC, this procedure resulted in 14 no. 8 (25M) black steel bars as the neck longitudinal reinforcement and no. 4 (13M) transverse hoops spaced at 2 in. (50 mm). The neck bars were inside the section, cut at the column-footing interface, and different from the exposed BRR. A 1.25 in. (31.8 mm) thick steel plate with a diameter matching that of the neck section was placed between the RPH-PC column and its footing to prevent concrete crushing at the rocking interface. The plate had a 6 in. (150 mm) diameter hole at the center to allow for the pipe to pass through the steel socket, which was embedded in the footing (Fig. 2).

The general construction sequence for all precast concrete columns was as follows: casting the column with self-consolidating concrete (SCC) (for RPH-PC) or UHPC (for RPH-PF and RPH-NP) at a precasting plant, casting the footing in the laboratory, shipping the precast concrete columns to the laboratory, installing the columns, and installing the exposed fuses. RPH-PC was constructed and tested prior to the construction of the other two precast concrete columns to learn from the first test and to potentially improve the other two columns' detailing.

The geometry and detailing of RPH-PF (Fig. 2) were mostly the same as those of RPH-PC, but with some modifications based on lessons learned from the first precast concrete column testing (RPH-PC). One major change in RPH-PF compared with RPH-PC was the use of UHPC in the precast concrete column instead of SCC to reduce the column damage. Furthermore, conventional black steel was used as the longitudinal reinforcement instead of stainless steel because the structural performance of stainless steel bars was explored in RPH-PC. Despite attempts to eliminate buckling of BRR in RPH-PC, a major Z-shaped bending was observed at high displacements outside BRR within the gap between the coupler and steel tube. To eliminate this bending, a new bar–to–steel tendon coupler was invented in collaboration with a leading coupler manufacturer in which the exposed fuses work in tension only in place of the tension-compression mechanism of BRR. The male portion of the couplers was redesigned to accommodate 0.6 in. (15 mm) diameter, seven-wire steel tendon. Because the area of a 0.6 in. tendon is approximately one-third that of the 1 in. (25 mm) diameter bar used in CIP and BRR of RPH-PC, the number of longitudinal reinforcing bars in RPH-PF was increased from 10 to 14 to better match the moment, and thus lateral force capacity, of CIP/RPH-PC. Another detailing modification was that the pipe (now a solid steel shaft) in RPH-PF was embedded into the footing rather than the column, which was the case in RPH-PC. This allowed for the steel socket to be inverted and placed in the column neck section, eliminating durability issues, such as possible water buildup in the steel socket, if used in real applications. Furthermore, the pipe cross section was changed from a circle to a solid square to better resist column torsion in biaxial loading. For both RPH-PC and RPH-PF, the pipe-pin design was based on the method proposed by Zaghi and Saiidi.[23]

PRH-NP was a hybrid rocking column version of RPH-PF. The cross-sectional properties, reinforcement, and cementitious materials of PRH-NP (Fig. 2) were the same as those of RPH-PF. The entire precast column consisted of UHPC. Steel tendons were incorporated as tension-only fuses and no BRR was used. The hybrid rocking column was connected to the footing using a grouted socket connection. The column socket connection was designed based on the recommendations of Saiidi et al.[24] The footing socket was constructed using a 30 in. (760 mm) diameter steel corrugated duct conforming to ASTM A760. The duct was capped at the bottom using a thin piece of sheet metal and rested on the bottom layer of the footing reinforcement. The 32 in. (810 mm) deep socket was confined with no. 4 (13M) transverse hoops spaced at 2 in. (50 mm). Eight no. 4 steel bars were placed diagonally to reinforce the socket at the four corners of the footing, in addition to the top layer of reinforcing mesh. Bars in the top layer of mesh that would otherwise intersect the socket had to be cut and bent. To accommodate the socket connection, the height of the footing was increased from 2 ft (0.6 m), used in all other columns, to 3 ft (0.9 m). However, the column height from the footing surface to the centerline of the applied lateral load was kept the same as other specimens at 8 ft (2.44 m) by shifting the actuator up. After securing the precast concrete column, the gap between the column and the corrugated duct was filled with a high-strength nonshrink grout at plastic consistency. Post-tensioning requirements of this hybrid rocking column were based on recommendations by Saiidi et al.,[3] in which the total tendon area must be greater than 0.4% of the gross cross-sectional area of the column and the initial tendon stress after all losses should be less than 30% of the tendon yield strength. These requirements ensure that post-tensioning steel tendons do not yield at hybrid rocking column failure (for example, core concrete crushing or fracture of energy dissipating bars). The final design resulted in fifteen 0.5 in. (13 mm) diameter, Grade 270 (1860 MPa), seven-wire steel strands, each post-tensioned to no more than 60 ksi (414 MPa). An anchorage system was incorporated to anchor the 15 unbonded tendons, which were grouped in a 3 in. (76 mm) plastic duct (Fig. 2) at the column center.

Reinforcement of the neck section of RPH-NP was kept the same as RPH-PF, with twelve no. 9 (29M) longitudinal bars and no. 4 (13M) double hoops spaced at 2 in. (50 mm). In RPH-PC and RPH-PF, the neck longitudinal bars were cut at the column-footing interface due to the dry connection. However, with the neck longitudinal bars extending into the lower portion of the column in RPH-NP (because the column was now to be connected to the footing through the socket connection), it was necessary to reduce the moment caused by the neck longitudinal bars. To this end, it was decided to use a reinforcing bar hinge at the column-footing level of RPH-NP to minimize the contribution of the neck bars to the overall moment capacity of the column. The reinforcing bar hinge was formed by bending the neck bars toward the center of the column at the column-footing level (Fig. 2). The neck double hoops continued below the reinforcing bar hinge.

Column test setup, instrumentation, and loading protocol

A setup with a cantilever configuration was used to laterally test the columns (Fig. 2). Four concrete blocks were stacked and post-tensioned to the lab strong floor as the reaction wall. A 328 kip (1460 kN) hydraulic actuator was mounted between the reaction blocks and the head of the column to apply the horizontal displacements. A self-reacting axial load system composed of high-strength threaded rods, two hollow-core jacks, and a spreader beam mounted on the top of the column was used to apply the column axial load.

Test data was recorded using various instruments. Steel strain gauges were installed on the longitudinal and transverse reinforcement at various levels within the footing and column cross sections (34 in cast-in-place concrete columns and 36 in the precast concrete columns). Furthermore, 10 concrete strain gauges were also used in a circular pattern at the base of the three precast concrete columns to evaluate concrete stresses at the rocking interface. In RPH-NP, five additional steel strain gauges were placed on the post-tensioning tendons to monitor their strains during tensioning and testing. Three string potentiometers were used to measure the horizontal displacements of the column head. The actuator load cell measured the column lateral load. Column rotations and curvatures were measured within the plastic hinge region using 10 linear variable displacement transducers (LVDTs), five on each loading face of the column. A 100 kip (440 kN) load cell was placed above each of the hollow-core jacks to measure the column axial load during testing. The axial load was adjusted at runs with large displacements to achieve a constant target axial load of 155 kip (689 kN). Data was recorded at a sampling rate of 10 Hz using a 128-channel data acquisition system.

The column specimens were tested under an increasing cyclic drift-based lateral loading protocol conforming to ACI 374.2R-13.[25] Drift is the ratio of the column lateral displacement to the column height. The target drifts were 0.25%, 0.5%, 0.75%, 1%, 2%, 3%, 4%, 5%, 6%, 7%, 8%, 9%, and 10%. The columns were subjected to two full cycles of each drift ratio. Initial cycles with drift ratios from 0.25% to 2% used a displacement rate of 3.0 in./min (76.2 mm/min) to capture the yield point of the columns. The postyield displacement rate was 10 times larger. These displacement rates were estimated based on ASTM E8 strain rate limits for testing steel bar. All columns were tested under the same loading protocol discussed herein but the precast concrete columns were tested twice to evaluate the proposed repair-by-fuse-replacement method.

Experimental results

Material properties

The materials used in the construction of the CIP and precast concrete columns were conventional ready-mixed concrete, SCC,

Table 2. Measured compressive strength of cementitious materials used in column models

Material	Element	Age, days	Compressive strength, psi						
			CIP	RPH-PC	RPH-PC-R	RPH-PF	RPH-PF-R	RPH-NP	RPH-NP-R
Concrete*	Footing	7	3670	5954	5954	6524	6524	6524	6524
		28	4620	6894	6894	n/a	n/a	n/a	n/a
		Test day	4920	7421	7250	6515	6515	6617	6631
	Column	7	3360	9161	9161	14,655	14,655	14,655	14,655
		28	4010	10,189	10,189	n/a	n/a	n/a	n/a
		Test day	4300	10,699	10,597	14,533	14,533	15,068	15,144
Grout	BRR and footing socket	7	n/a	6132	6132	n/a	n/a	n/a	n/a
		28	n/a	n/a	n/a	n/a	n/a	n/a	n/a
		Test day	n/a	7055	7568	n/a	n/a	6845	7460

Note: BRR = buckling-restrained reinforcement; CIP = cast in place concrete column; n/a = not applicable; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-NP-R = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection repaired by fuse replacement and retested; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PC-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base repaired by fuse replacement and retested; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing; RPH-PF-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing repaired by fuse replacement and retested. 1 psi = 6.895 kPa.

* Conventional concrete was used in the cast-in-place concrete column and all footings. Self-consolidating concrete was used in the RPH-PC column. Ultra-high-performance concrete was used in the RPH-PF and RPH-NP columns.

UHPC, nonshrink grout, stainless and conventional steel bars, and steel tendons. The mechanical properties of each of these materials were measured following their standard procedures.

Conventional concrete from a local ready-mixed concrete plant was used in the construction of all footings as well as the cast-in-place concrete column. ASTM C39 testing protocol was used to measure the compressive strength of standard 6 in. (150 mm) diameter cylinder samples. **Table 2** presents the average measured concrete strength at 7 days, 28 days (when applicable), and the column test day.

The RPH-PC column was poured using SCC, as it is a typical material in the precast concrete industry. Standard samples were taken and tested according to ASTM C39. Table 2 also presents a summary of SCC strength on different days.

UHPC was used in the construction of the RPH-PF and RPH-NP columns. The UHPC mixture was designed by the precasting plant using local materials. Samples were taken using 3 in. (76 mm) diameter cylinders, which is a common sampling size for UHPC due to its high strength. UHPC samples were cut using a diamond-blade saw to provide a flat surface and to avoid point load under compressive loading. A summary of UHPC compressive strength is reported in Table 2. Due to time limitations in finishing the experimental part, the columns were tested prior to 28 days. The RPH-PF and RPH-PF-R columns were tested 9 days after both the column and footing placement, and columns RPH-NP and RPH-NP-R were respectively tested 16 and 17 days after the pour.

A high-strength, nonshrink grout was used in BRR fuses of RPH-PC and the grouted socket connection of RPH-NP. Two-inch (50 mm) cube samples were collected following ASTM C109, and their compressive strength was determined. Table 2 presents a summary of the measured grout compressive strength. Columns RPH-NP and RPH-NP-R were respectively tested three days and four days after the socket grout was placed, resulting in a lack of 7- and 28-day strength data for the grout in the table.

Both conventional black steel bars conforming to ASTM A706 Grade 60 (414 MPa) and stainless steel bars conforming to ASTM A955 Grade 60 (414 MPa) were used in this project. The CIP column was longitudinally reinforced using no. 8 (25M) conventional steel bars. The three repairable columns were longitudinally reinforced using no. 10 (32M) bars (stainless steel in RPH-PC; conventional black steel in RPH-PF and RPH-NP). The BRR fuses used in RPH-PC were stainless steel with a dog-boned diameter of 1.0 in. (25 mm). Nevertheless, 0.6 in. (15 mm) diameter steel tendon fuses were used in RPH-PF and RPH-NP. Furthermore, the post-tensioning of the rocking RPH-NP column was provided using fifteen 0.5 in. (13 mm) diameter steel tendons. All tendons used in this project conformed to ASTM A416 Grade 270 (1860 MPa). The transverse reinforcement of all four specimens was no. 4 (13M) hoops. ASTM E8 was followed for the tensile testing of all steel reinforcement,

Table 3. Measured strength of steel reinforcement used in column models

Bar	Column model	Bar size	ASTM type	Yield strength, ksi	Ultimate strength, ksi	Postyield stiffness, ksi	Ultimate strain, %*
Longitudinal bars	CIP	No. 8	A706 Grade 60	69.3	97.4	853	12.0
	RPH-PC and BRR fuses	No. 10	A955 Grade 60	90.4	113.4	930	16.6
	RPH-PF	No. 10	A706 Grade 60	67.1	105.5	1125	11.7
	RPH-NP	No. 10	A706 Grade 60	67.1	105.5	1125	11.7
Tendons as fuses	RPH-PF and RPH-NP†	0.6 in.	A416 Grade 270	284	306.5	863	6.2
Hoops	CIP	No. 4	A706 Grade 60	66.6	102.1	1873	9.9
	Repairable precast concrete	No. 4	A706 Grade 60	65.3	100.7	2567	9.8

Note: BRR = buckling-restrained reinforcement; CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing. Grade 60 = 414 MPa; Grade 270 = 1860 MPa; no. 4 = 13M; no. 8 = 25M; no. 10 = 32M. 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

* Strain at the peak stress.

† Post-tensioning tendons in RPH-NP were not tested. ASTM A416 provides strength values.

and **Table 3** presents a summary of the average measured properties.

Observed damage

As discussed previously in the section "Column Test Setup, Instrumentation, and Loading Protocol," all columns were tested laterally in a systematic manner to investigate their performance. CIP was tested to failure in one round. However, the precast concrete columns were tested twice. The first round of testing of the precast concrete columns stopped at either 5% drift ratio (for RPH-PC) or 4% drift ratio (for RPH-PF and RPH-NP). Subsequently, the external fuses of each precast concrete column were replaced with new ones and the column was retested to failure (marked with R). The repair was done by just replacing fuses; no other repair method, such as concrete patching or jacketing, was used.

Figure 3 shows the damage of the plastic hinge region for CIP, RPH-PC, RPH-PF, and RPH-NP at the end of the 2% drift cycle. CIP and RPH-PC experienced some flexural cracks, while no damage was observed in RPH-PF and RPH-NP. The observed damage for CIP was typical as it was a conventional column. Some cracks were expected in RPH-PC as it was cast with SCC. At the rocking face of RPH-PC, some spalling was also observed, mainly due to the use of steel shims placed between the column and its steel baseplate. The apparent damage of RPH-PF and RPH-NP, both cast with UHPC, was insignificant at 2% drift ratio.

Figure 4 shows the damage of the four columns at 4% drift ratio (just prior to replacing the fuses of the precast concrete columns). CIP experienced significant spalling at this drift level. RPH-PC exhibited both flexural and shear cracks above the neck section, a minor Z-shaped bending of BRR, and spalling of SCC at the column base. RPH-PF showed no damage and RPH-NP exhibited minor flexural cracks in the neck section.

Figure 5 shows the plastic hinge damage of CIP and the three repaired columns at their failure displacement. CIP exhibited extensive damage, such as concrete crushing and buckling and fracture of its longitudinal bars. Several cracks, spalling of SCC, and a Z-shaped bending of BRR occurred in RPH-PC-R. RPH-PC-R did not fail at 10% drift ratio, but the test was stopped, matching the CIP failure cycle. RPH-PF-R had only a few cracks at the base of the column. RPH-NP-R had a few more flexural cracks than RPH-PF-R both on and above the neck section. The failure of RPH-PF-R was due to the rupture of three tendon fuses during 8% and 9% drift cycles. Furthermore, RPH-NP-R failed due to a significant strength degradation caused by the reinforcement fracture within the reinforcing bar hinge during 8% drift cycles. Overall, the damage to the two UHPC columns (the body of the RPH-PF and RPH-NP columns) was insignificant at the column failure. Ruptured tendons can be replaced multiple times but were replaced once in this study.

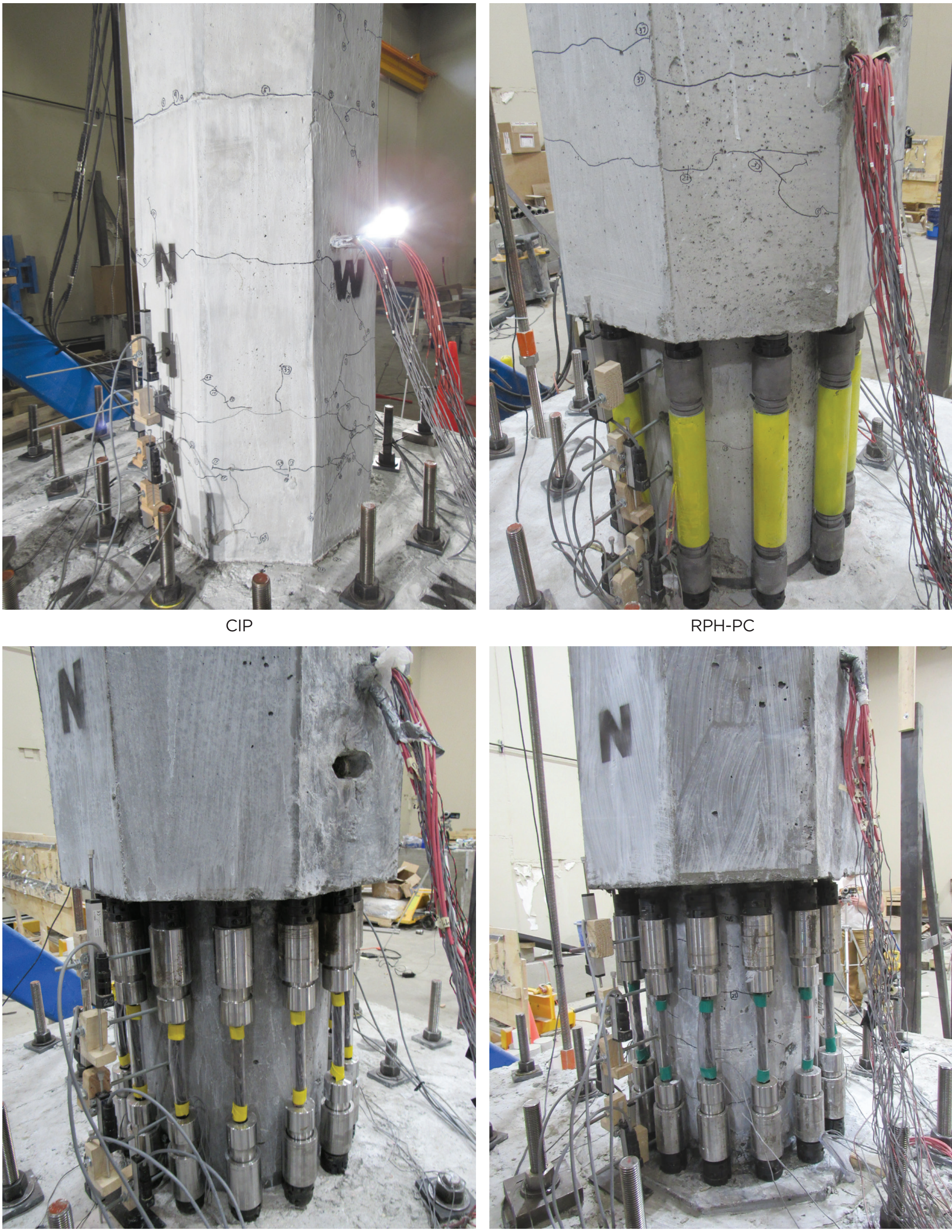
Force-displacement relationships

Figure 6 shows the measured lateral force-drift hysteretic response for the three precast concrete columns, including their repaired versions superimposed on the CIP response. The column effective stiffness was the ratio of the yield force to the yield displacement, as defined in AASHTO.[20] The yield point was where the first longitudinal bar (steel bar in in the cast-in-place concrete column or stainless steel buckling-restrained reinforcement or tendon fuse in the precast concrete columns) yielded in tension. The column displacement capacity was the point at which the column lateral-load-carrying capacity dropped by 15% compared with its peak lateral strength, or where the test was stopped (for RPH-PC-R).

The CIP effective stiffness was 88.4 kip/in. (15,484 kN/m), and the CIP drift capacity was 8.96%. RPH-PC and RPH-PC-R exhibited 65% and 80% smaller effective stiffness, respectively, compared with that of CIP. RPH-PC-R had 11% higher strength and 9% larger displacement capacity compared with those of CIP (Fig. 6). In cyclic testing of bridge columns, residual displacements are the column lateral displacements during unloading at zero forces. Residual drifts are the ratio of the column residual displacements to the column length. Though no specific self-centering mechanism was used, RPH-PC and its repaired version, both with stainless steel BRR, showed smaller residual displacements than CIP (for example, 44% smaller residual displacement at the last cycle). However, this residual displacement reduction was not sufficient to allow an easy BRR replacement. The RPH-PC column and footing dowel bars were bent at high drifts (Fig. 5), which limited the repair-by-replacement technique using BRR. More discussion on the repairability of the proposed precast concrete columns is provided in the section "Repairability, inspectability, and durability of proposed precast concrete columns."

RPH-PF and RPH-PF-R showed 85% lower effective stiffness, 11% lower lateral strength, and similar displacement capacity compared with CIP (Fig. 6). The lower strength of RPH-PF was due to the use of 14 tendon fuses compared with 10 steel bars of CIP. A higher lateral strength could be achieved using a larger tendon size or a higher number of longitudinal reinforcement. Both RPH-PF and RPH-PF-R exhibited insignificant residual displacements throughout the entire testing and retesting (a flagged shape hysteresis), which allowed a simple and quick replacement of tendon fuses.

For RPH-NP and RPH-NP-R, the initial stiffness was the highest among all three precast concrete columns, but smaller than that of CIP (Fig. 6). The effective stiffness of PHF-NP and RPH-NP-R was 75% and 80% smaller than that of CIP, respectively. Furthermore, PHF-NP showed a 33% higher lateral strength and a 14% lower displacement capacity compared with CIP. The RPH-NP-R failure at 7.7% drift ratio was because of multiple bar fractures within the reinforcing bar hinge. The higher strength was due to the reinforcement of the


CIP

RPH-PC

RPH-PF

RPH-NP

Figure 3. Column specimen plastic hinge damage at 2% drift cycle. Note: CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing.

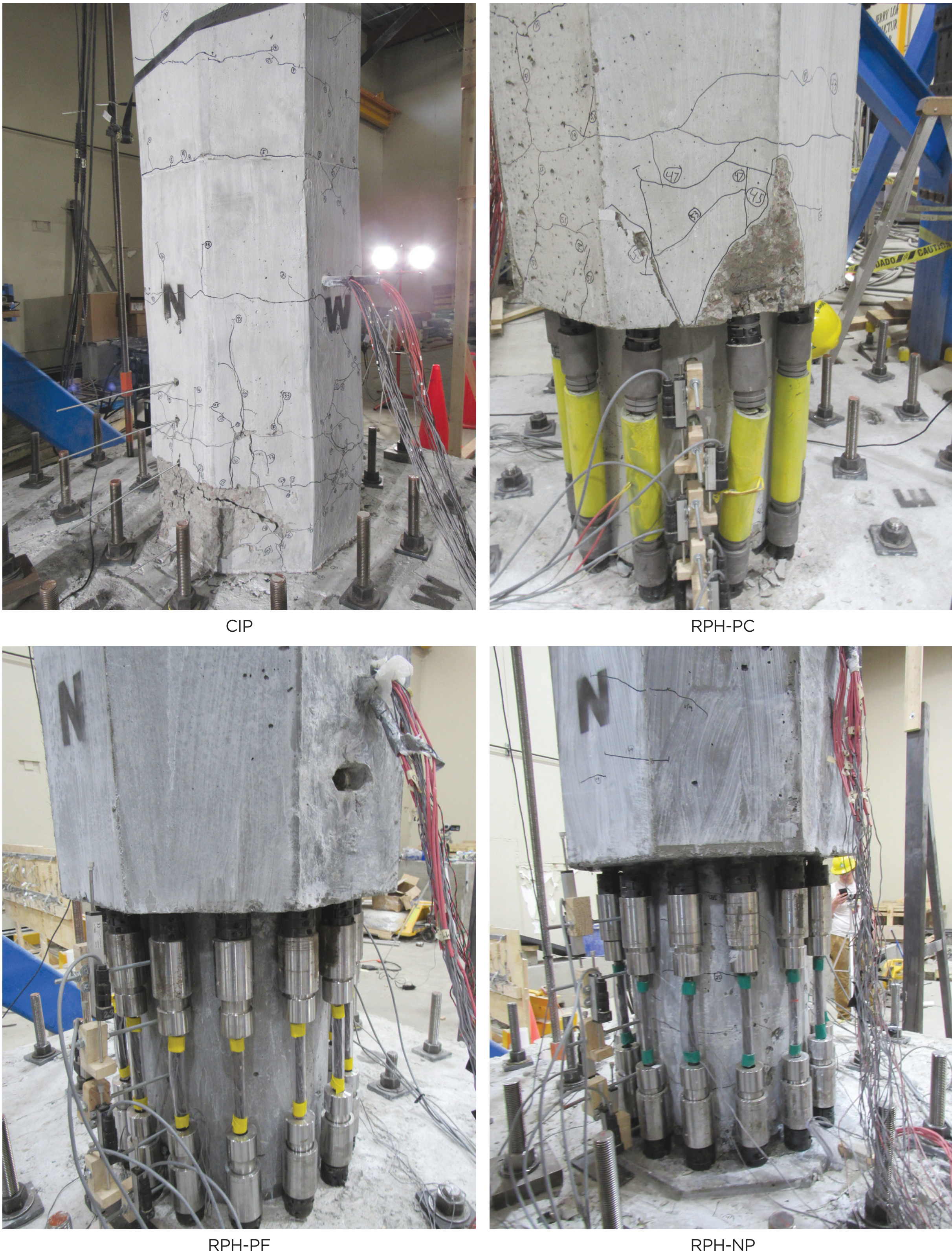


Figure 4. Column specimen plastic hinge damage at 4% drift cycle. Note: CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing.

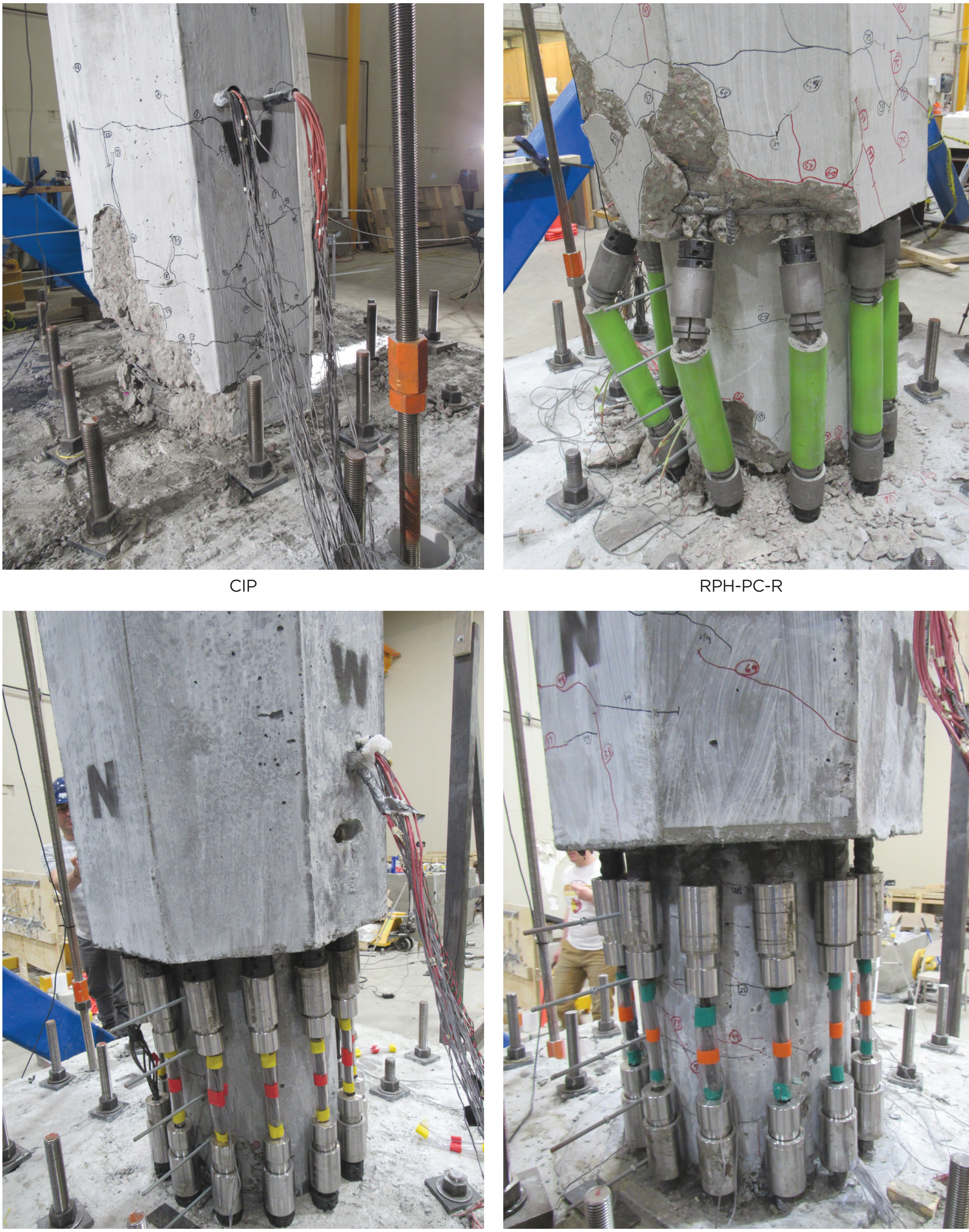


Figure 5. Column specimen plastic hinge damage at failure state. Note: CIP = cast-in-place concrete column; RPH-NP-R = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection repaired by fuse replacement and retested; RPH-PC-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base repaired by fuse replacement and retested; RPH-PF-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing repaired by fuse replacement and retested.

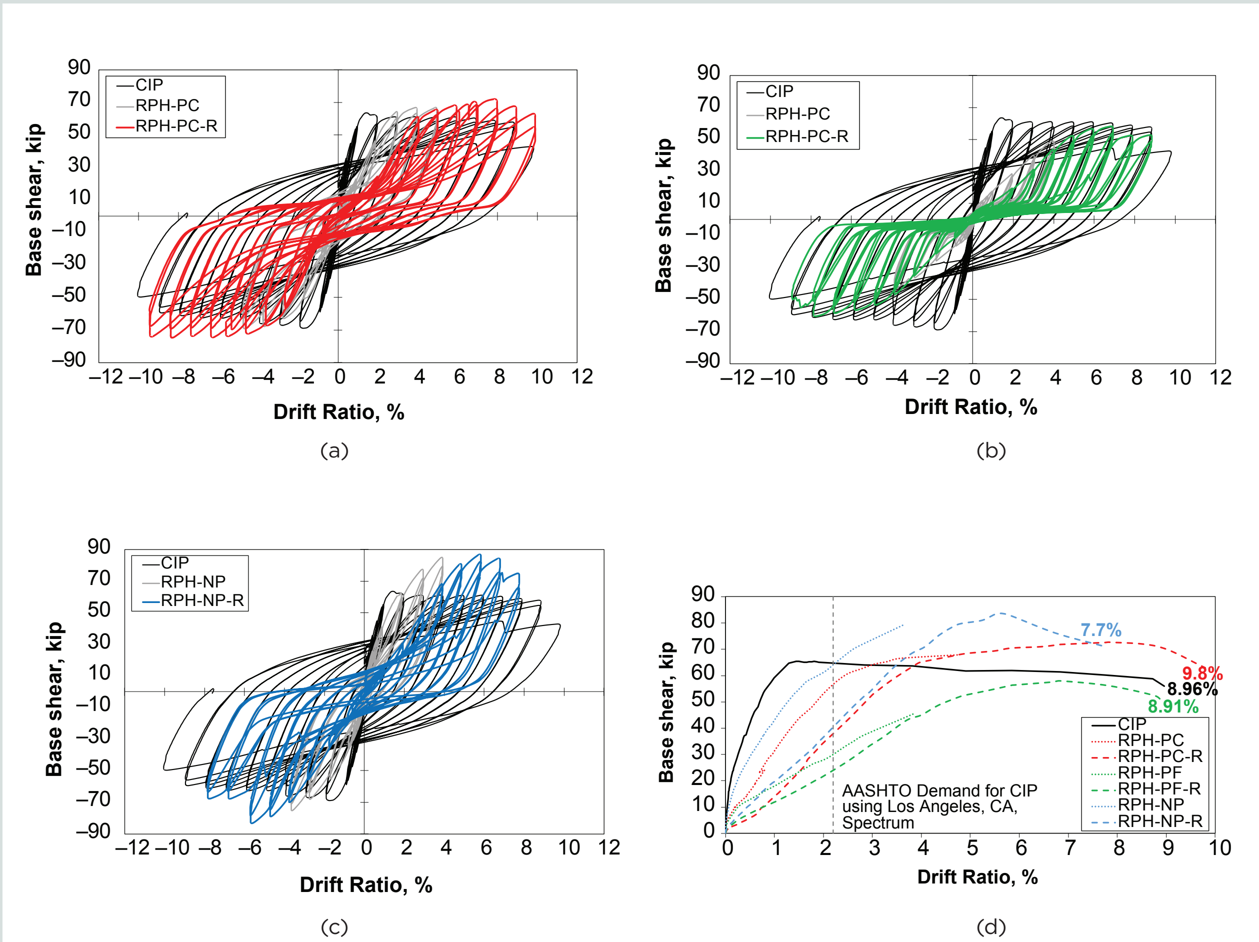


Figure 6. Measured force-drift hysteretic and envelope responses for the column specimens. Note: CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-NP-R = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection repaired by fuse replacement and retested; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PC-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base repaired by fuse replacement and retested; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing; RPH-PF-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing repaired by fuse replacement and retested. 1 kip = 4.448 kN.

reinforcing bar hinge and the internal post-tensioning tendons. Despite the use of the conventional rocking mechanism (the unbonded internal post-tensioning tendons) as well as the external tendon fuses, the column residual displacements were relatively large, mainly due to the bar yielding within the reinforcing bar hinge at the column base. Though RPH-NP exhibited 1.5% residual drift ratio after 4% peak drift cycles, the repair of this column was also easy and quick because the new tendon-to-bar coupler provided a large tolerance, and tendons were cut with the new required lengths.

Figure 6 shows the average pushover envelopes for the CIP and all precast concrete columns. The figure also includes the design level drift demand based on the AASHTO design spectrum for downtown Los Angeles, Calif., using the effective period of CIP. This site has a seismic design category of D, which is the highest in AASHTO. All precast concrete columns were pushed beyond the CIP displacement demand and were successfully repaired by the fuse replacement.

Residual displacements

Figure 7 shows the measured residual-versus-peak drift relationships for the CIP and precast concrete columns. All columns had a negligible residual displacement up to 1% peak drift. Furthermore, RPH-PC and RPH-PF exhibited insignificant residual drifts up to 4% peak drift. The residual-to-peak drift relationships for RPH-PF-R and RPH-NP-R closely matched those of their initial testing, while RPH-PC-R experienced smaller residual drifts compared with its initial testing (RPH-PC). After their initial plateaus, the residual-to-peak drift response for RPH-PC-R and RPH-NP-R followed the same slope as CIP, while the response for RPH-PF-R increased at a smaller rate. Residual drifts less than 1%

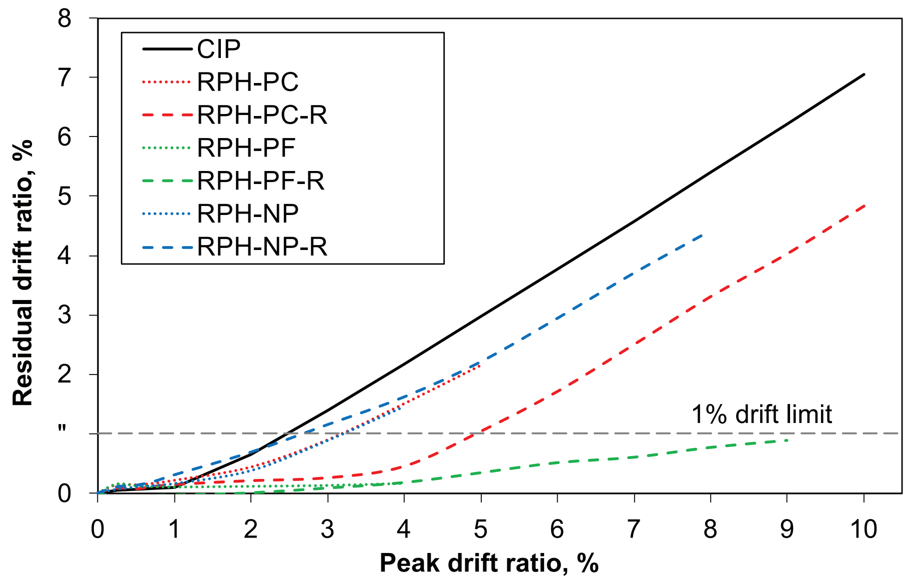


Figure 7. Measured column specimen residual drifts. Note: CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-NP-R = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection repaired by fuse replacement and retested; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PC-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base repaired by fuse replacement and retested; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing; RPH-PF-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing repaired by fuse replacement and retested.

may be assumed insignificant after an earthquake.[3] Overall, RPH-PF-R experienced the smallest residual drifts among all columns and stayed below the 1% residual drift limit.

Self-centering is a key parameter in the repairability of the proposed precast concrete columns because a close-to-plumb position allows for a quick and easy replacement of the fuses. RPH-PF exhibited the best self-centering among all precast columns, even higher than the one with the conventional rocking detailing (RPH-NP). The self-centering mechanism of RPH-PF was achieved through external tendon fuses, which were fully accessible and replaceable.

Strain profiles

Figure 8 shows the maximum measured tensile strains at each strain gauge level for the cast-in-place and the three precast concrete columns. Each longitudinal bar or fuse of the columns was labeled (Fig. 2) and the strains of those at the same location or the nearest one were compared. In the precast concrete columns, the center two levels of strain gauges were placed on the dog-bone stainless steel buckling-restrained reinforcement of RPH-PC and RPH-PC-R, and on the tendon fuses of RPH-PF, RPH-PF-R, RPH-NP, and RPH-NP-R. The outer two levels were placed on the column and footing dowels. The highest strains were at the fuses of the precast concrete columns (Fig. 8), indicating a successful fuse design using either the reduced bar diameter in BRR or small tendons. Furthermore, the strains of column-footing dowel bars in the two tendon fuse columns (RPH-PF and RPH-NP) were insignificant and much lower than those of CIP and RPH-PC-R. Overall, all fuses (either stainless steel BRR or steel tendon) were indeed the weak link in the columns, exhibiting significant strains several times those of their adjoining column-footing dowel bars.

Steel strain gauges placed on the neck longitudinal reinforcement in all precast concrete columns showed mostly linear-elastic behavior even at the column failure, indicating a successful design for the neck, which was discussed in the section "Design and Construction of Test Specimens." The compressive strains of SCC in RPH-PC and UHPC in RPH-PF remained below 5000 $\mu\varepsilon$ indicating that concrete had minimal spalling at the rocking interface. However, the UHPC compressive strain in RPH-NP exceeded 8000 $\mu\varepsilon$. The higher concrete compressive strain of RPH-NP compared with other precast concrete columns was likely due to its lateral strength which was the highest among all columns.

The maximum tensile strain of the RPH-NP post-tensioning tendons (internal tendons) at the column failure, including all losses and the column axial load, was 3115 $\mu\varepsilon$, which was only 36% of the tendon yield strain. Overall, the internal post-tensioning tendons of the RPH-NP column did not yield

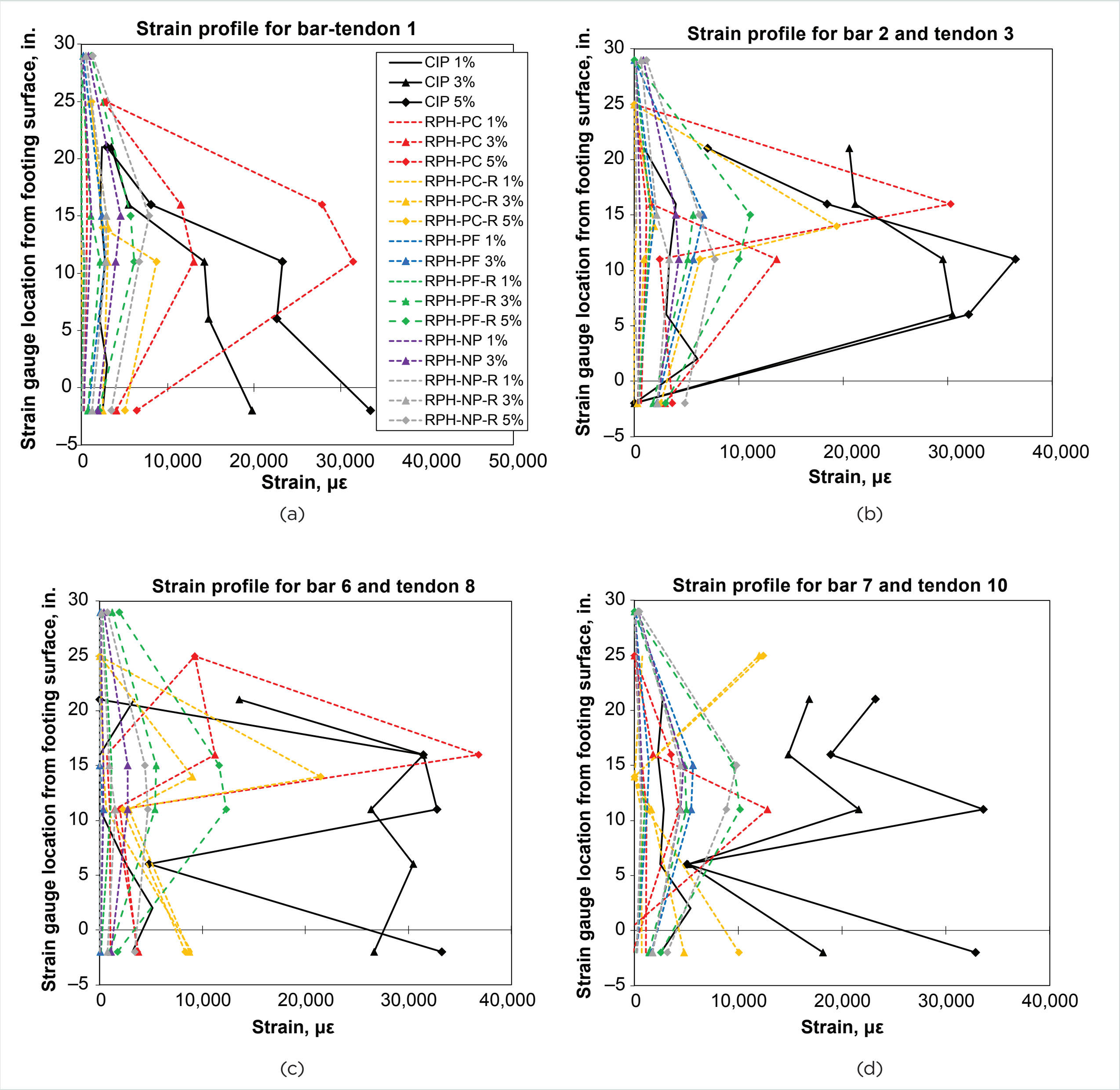


Figure 8. Measured cast-in-place and precast concrete column tensile strain profiles. Note: CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-NP-R = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection repaired by fuse replacement and retested; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PC-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base repaired by fuse replacement and retested; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing; RPH-PF-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing repaired by fuse replacement and retested. 1 in. = 25.4 mm.

during the entire testing and retesting validating its design as discussed in the section "Design and Construction of Test Specimens."

Energy dissipation

Figure 9 shows the cumulative energy dissipation of the four columns. All precast concrete columns dissipated significantly lower energy than the cast-in-place concrete column. At the same drift level, the second test of each precast concrete column also exhibited lower energy dissipation than the original specimen, mainly due to the accumulation of damage. RPH-PF-R experienced the lowest energy dissipation among all columns due to the flag-shaped hysteresis caused by the recentering of the tendon fuses. Furthermore, RPH-NP-R exhibited a larger energy dissipation than the

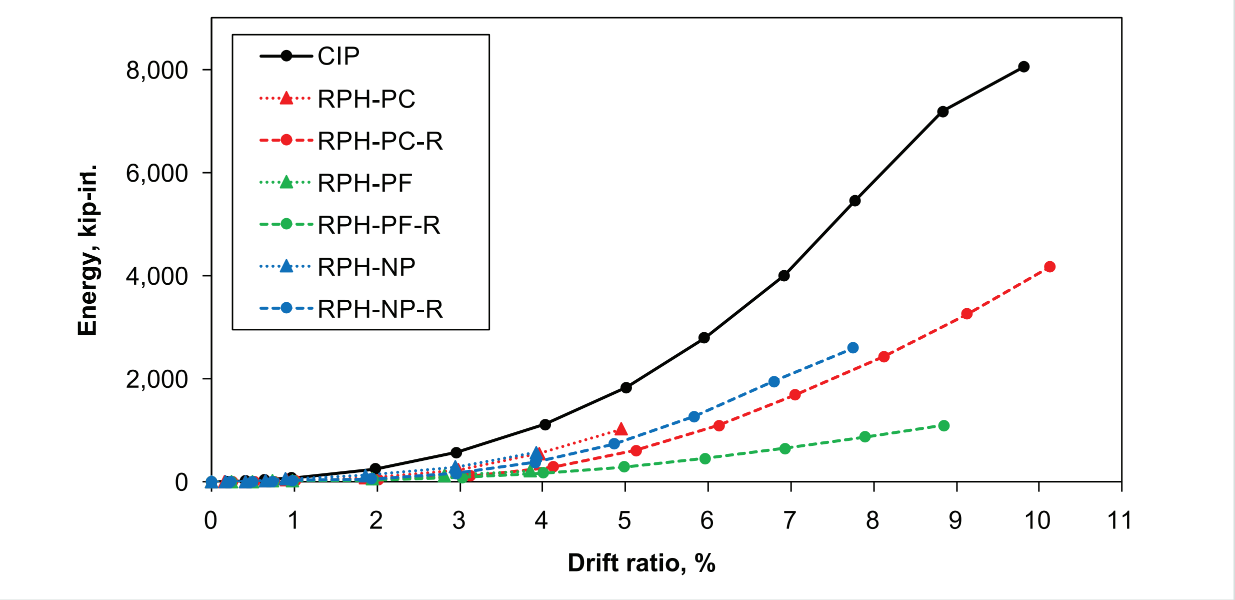


Figure 9. Energy dissipation for the column specimens. Note: CIP = cast-in-place concrete column; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-NP-R = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection repaired by fuse replacement and retested; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PC-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base repaired by fuse replacement and retested; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing; RPH-PF-R = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing repaired by fuse replacement and retested. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

other precast concrete columns due to the yielding of the longitudinal reinforcement of the reinforcing bar hinge connection incorporated in the column at the column-footing level.

Summary of experimental study

Table 4 presents a summary of the key experimental data measured in the four columns. Overall, all precast concrete columns exhibited approximately the same force and displacement capacities compared with the cast-in-place concrete column. However, the effective stiffness of the precast concrete columns was only 15% to 35% that of the cast-in-place concrete column. The lower stiffness of these columns will result in a longer natural period (for example, 0.423 seconds for CIP versus 1.113 seconds for RPH-PF) and thus a higher displacement demand under seismic actions. Using the AASHTO design spectrum for Los Angeles, Calif., the drift demand of RPH-PF, the softest column, was 2.5 times larger than that of CIP, or 5.6% drift ratio. Each precast concrete column exhibited a drift capacity (Fig. 6) that was significantly higher than its design demand (for example, 8.91% drift capacity versus 5.6% drift demand for RPH-PF). Further experimental investigation is needed to better understand the dynamic behavior of the proposed precast concrete columns with repairable detailing.

Repairability, inspectability, and durability of proposed precast concrete columns

In terms of repairability, repair of CIP was impractical at 10% drift ratio due to the extent of damage. Some repair techniques are available for reinforced concrete columns with extensive damage, such as replacing fractured bars[26] and shifting the plastic hinge location;[27] however, they were not used in this project.

The repair of RPH-PC was difficult after 5% drift cycles due to the bending of BRR and the column and footing dowel bars (Fig. 4), and 1.5% residual drifts (Fig. 7). BRR were fabricated prior to testing, so replacing them when the column was tilted was difficult. The actuator was used to push the column back to its original position to replace BRR. This would be impractical in real applications. Therefore, the proposed precast concrete columns reinforced with buckling-restrained reinforcement, such as RPH-PC, are not truly repairable through component replacement.

RPH-PF had insignificant residual displacements with no damage of the column and footing dowel bars, so it was easy to repair. The repair was done by removing the existing tendons through detaching the coupler pieces, cutting new tendons, placing them inside the couplers, and torquing the coupler with

Table 4. Summary of column test results

Column model	Initial stiffness,* kip/in.	Effective stiffness, kip/in.	Effective stiffness after repair, kip/in.	Yield force, kip	Ultimate drift ratio, %	Peak lateral force, kip	Mode of failure
CIP	304.0	88.4	n/a	38.2	8.96	65.4	Longitudinal bar fracture
RPH-PC	60.2	30.5	18.0	43.0	9.80	72.7	Did not fail
RPH-PF	45.9	12.8†	11.7	47.5	8.91	57.9	Tendon fuse rupture
RPH-NP	163.1	22.2†	17.5	79.1	7.68	86.9	Bar fracture in reinforcing bar hinge

* Initial stiffness is not used in bridge engineering. However, this stiffness, which was defined as the force-to-displacement ratio corresponding to 8.5 kip, was included to better compare the stiffnesses of different columns. Data was available at this point for all columns.

† No yielding was observed where the test was stopped (at 4% drift ratio). This stiffness was the force-to-displacement ratio at the end point.

Note: CIP = cast-in-place concrete column; n/a = not applicable; RPH-NP = repairable precast concrete column with headed longitudinal bars/couplers and no pin (socket) connection; RPH-PC = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the column at the column base; RPH-PF = repairable precast concrete column with headed longitudinal bars/couplers and pipe-pin connection embedded in the footing. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

pipe wrenches. The entire repair process, including strain gauging of the new fuses and the column retest setup preparation, took less than two hours. RPH-PF and RPH-PF-R were tested on the same day. Similarly, the repair of RPH-NP was simple and quick after 4% drift cycles with 1.5% residual drift ratio. Even at this residual displacement, the tendons were cut to the new lengths and installed. This is a significant enhancement of repairability compared with RPH-PC, where BRR lengths cannot be easily adjusted on-site (a portable heading or threading machine is needed to prepare the ends of BRR on-site), and there is a small tolerance to install each BRR.

Furthermore, tightening the couplers using simple tools (pipe wrenches) generated a significant tensile force in tendon fuses of RPH-PF and RPH-NP (as much as 10 kip [44 kN]), so such fuses may serve as both the column longitudinal reinforcement and a recentering mechanism. Therefore, the proposed detailing, especially RPH-PF, is a novel rocking connection in which post-tensioning is outside the column and fully accessible. This is a significant enhancement of inspectability compared with conventional rocking systems because post-tensioning of rocking columns in all past studies was internal, with no to minimal accessibility for inspection and maintenance.

Exposed tendons and dowel bars will cause durability issues. To address this concern, all exposed reinforcement should be protected against weather conditions. For example, stainless steel bars must be used as column-footing dowel bars; and stainless steel tendons (less ductile than conventional steel tendons), galvanized tendons, or epoxy-coated tendons (such as those used in cable-stayed bridges, which may need special fixtures) should be incorporated as tendon fuses. The neck region can be covered with a casing matching the column section and color to prevent vandalism. UHPC, which is a durable material, showed superior seismic performance compared with conventional concrete and SCC, so UHPC is recommended in repairable columns.

Overall, the two precast concrete columns with UHPC and tendon fuses were easily and quickly repaired by fuse replacement using simple tools. Therefore, RPH-PF and RPF-NP are proposed as repairable columns.

Conclusion

One cast-in-place and three precast concrete columns with replaceable fuses, all at 50% scale, were tested under the same cyclic loading protocol to failure. The precast concrete columns were tested twice to practice a repair-by-fuse-replacement technique. The first round of testing of the precast concrete columns ended at either a 4% or 5% drift ratio to replace the exposed fuses. The external detachable fuses were made of either buckling-restrained reinforcement or steel tendons. Following is a summary of the experimental findings:

- CIP failed by the longitudinal bar fracture after extensive concrete spalling. The lateral strength of CIP was 65.4 kip (291 kN) with a drift capacity of 8.96%.
- Testing of RPH-PC-R was stopped at 10% drift cycle, matching the CIP failure cycle. SCC spalled above the neck section in RPH-PC-R, which was mainly due to the compressive forces of buckling-restrained reinforcement. Furthermore, a Z-shaped bending of buckling-restrained reinforcement was seen, which limited its replacement.

The lateral strength of RPH-PC-R was 72.7 kip (323 kN), which was 11% higher than that of CIP. The drift capacity of RPH-PC-R was 9.80%, which was 9% higher than that of CIP.

- RPH-PF-R failed by rupture of multiple tendon fuses with minimal UHPC damage. The lateral load capacity of RPH-PF-R was 57.9 kip (258 kN), which was 11% less than that of CIP. The drift capacity of RPH-PF-R was 8.91%, which was close to that of CIP.

- RPH-NP-R failed by strength degradation, mainly due to the rupture of the neck bars at the reinforcing bar hinge. Minor UHPC spalling at the column base was observed. The lateral load capacity of RPH-NP-R was 86.9 kip (387 kN), which was 33% higher than that of CIP. The drift capacity of this column was 7.68%, which was 14% less than the CIP drift capacity.

- The Z-shaped bending of stainless steel buckling-restrained reinforcement used in the RPH-PC column made repair by replacement difficult. Furthermore, the column and footing dowels of RPH-PC were bent and the column had some residual displacements, which further complicated the placement of new buckling-restrained reinforcement. The incorporation of the steel tendons as the tension-only fuses in RPH-PF and PPH-NP eliminated any column and footing dowel bar damage and further helped with the recentering of the columns. The replacement of the tendon fuses was simple and quick.

- Small residual displacements are desired in any repairable column because the column should be close to its original position, easing the fuse replacement. All precast concrete columns showed smaller residual displacements compared with cast-in-place concrete. However, the residual displacement of RPH-PF was insignificant throughout the testing and retesting.

- A tendon fuse can serve as both typical longitudinal reinforcement and self-centering reinforcement, with the additional benefit of full accessibility.

- Use of UHPC greatly reduced the precast concrete column damage, even at the fuse failure.

Overall, the three proposed alternatives with replaceable fuses were found feasible. However, the repair by fuse replacement was simple, quick, and easy when tension-only members were used as fuses. Further studies, such as shake-table testing, are needed to fully understand the dynamic behavior of repairable columns.

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About the authors

Mostafa Tazarv, PhD, PE, is an associate professor in the department of Civil and Environmental Engineering at South Dakota State University in Brookings, S.Dak. His research interests include seismic performance of concrete structures, large-scale testing, and accelerated bridge construction.

Kallan Hart is a former MS student in the department of Civil and Environmental Engineering at South Dakota State University. He is currently a structural engineer at DCI Engineers in Missoula, Mont.

Abstract

In multispan bridges excited by ground shaking, columns are usually the target ductile members. Although current practice is successful in attaining the no-collapse objective for bridges, columns are usually damaged at displacements associated with this performance level. Minor damage can be repaired, but excessive damage is usually beyond repair. A new design paradigm with minimal damage and repair need is gaining interest. The benefits of such a design can be increased if low-damage technologies are combined with accelerated bridge construction techniques. The main goal of the present study was to develop reinforced concrete bridge columns that are fully precast concrete, low damage, and repairable through component replacement. To achieve the project objectives, 20 repairable precast concrete alternatives were developed and ranked. Subsequently, the top three candidates were designed at 50% scale, constructed at a precast concrete plant, and tested in a laboratory. The precast concrete columns incorporated different exposed fuses, such as stainless steel bars and steel tendons, advanced materials such as ultra-high-performance concrete, a self-centering mechanism, or an accelerated bridge construction socket connection. A reference cast-in-place concrete column was included for comparison. Each precast concrete column was tested twice. It was found that the repair of the precast ultra-high-performance concrete columns with exposed tendons was easy, simple, and quick due to insignificant damage and minimal residual displacements. All precast concrete columns exhibited displacement capacities comparable to or higher than cast-in-place concrete columns.

Keywords

ABC, accelerated bridge construction, external post-tensioning, fuse, repairable columns, UHPC, ultra-high-performance concrete.

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