Executive Summary

1. Introduction

Accelerated bridge construction (ABC) has become increasingly popular throughout the United States because of its numerous advantages. In many cases, ABC methodologies have been shown to decrease bridge construction time, reduce the overall project cost, and reduce the impact on the environment and traveling public. To effectively execute ABC projects, designers use prefabricated structural elements that can be manufactured offsite in parallel with on-site construction, which can result in improved element quality. These members are then delivered to the site and can be quickly assembled to form a functional structural system. Despite the numerous advantages, ABC has not been extensively used in areas subject to moderate and high seismic hazards for good reason. There is a great deal of uncertainty about the seismic performance of the connections used to join precast elements. Of specific concern are substructure connections (column-footing, column-shaft, and column-bent-cap) because they must dissipate energy through significant cyclic nonlinear deformations under seismic loading while maintaining their capacity and the integrity of the structural system.

The main objective of this study was to develop, test, analyze, and evaluate precast column-footing connections for ABC in moderate and high seismic zones. Unlike the majority of connections tested by previous researchers, which could require analysis or design considerations that deviate from conventional systems, the goal of this study was to develop connections that closely resembled conventional cast-in-place systems with respect to design, detailing, and performance. That is, the connections were to be emulative of conventional cast-in-place construction such that designer would not require specialized design methods or analysis. To achieve emulative detailing, mechanical reinforcing bar splices were used to connect precast columns to cast-in-place footings. A generalized comparison between conventional connections and the proposed mechanically-spliced precast column-footing connection is shown in Fig. 1.

![Figure 1 Comparison between conventional connection details and mechanically-spliced connections](image)

There were three main components to the investigation: 1) half-scale column testing, which consisted of the design, construction, and testing of five half-scale column models under reversed slow cyclic loading, 2) experimental testing of individual mechanically-spliced bars, which included static and dynamic tensile loading, single- and multi-cycle elastic slip testing, and cyclic loading tests, and 3) extensive analytical studies, which included developing OpenSEES models for the half-scale columns tested and prototype-scale models for parametric studies and development of design recommendations.

2. Mechanical Reinforcing Bar Splices and Selection Criteria
Most building and bridge seismic design codes have provisions that place minimum performance requirements on mechanical reinforcing bar splices. Usually in the form of specified stress or strains that must be achieved prior to failure, these performance standards constrain the application of the device depending on the expected demand. Table 1 outlines the current US code requirements for mechanically-spliced reinforcing bars.

Table 1 US design code requirements for mechanical reinforcing bar splices

<table>
<thead>
<tr>
<th>Code</th>
<th>Splice Designation</th>
<th>Stress Criterion for Spliced Bar</th>
<th>Strain Criterion for Spliced Bar</th>
<th>Maximum Slip Criterion</th>
<th>Location Restriction</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI318</td>
<td>Type 1</td>
<td>1.25(f_y)</td>
<td>none</td>
<td>none</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>Type 2</td>
<td>1.0(f_y)</td>
<td></td>
<td></td>
<td>No</td>
</tr>
<tr>
<td>AASHTO</td>
<td>Full-mechanical connection (FMC)</td>
<td>1.25(f_y)</td>
<td>none</td>
<td>Bar No. 3-14 = 0.01&quot; 18 = 0.03&quot;</td>
<td>No</td>
</tr>
<tr>
<td>Caltrans SDC</td>
<td>Service</td>
<td>Minimum Capacity 6% for No. 11 and larger 9% for No. 10 and smaller</td>
<td>Maximum Demand &lt; 2%</td>
<td>7-9 = 0.014&quot; 10-11 = 0.018&quot; 14 = 0.024&quot; 18 = 0.03&quot;</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>Ultimate</td>
<td>none</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: \(f_y\) - Specified yield strength of the spliced reinforcing bar
\(f_u\) - Specified tensile strength of the spliced reinforcing bar
1" = 25.4 mm

Two mechanical splices were selected for this study based on literature review and discussion with the sponsor, the California Department of Transportation (Caltrans). A number of different splices were initially considered. The factors that affected the final selection were Caltrans prequalification, applicability of splices to rapid installation, and consistent mechanical performance reported in the literature. Figure 2 shows the two coupler devices that were selected. The up-set headed coupler (HC) creates connectivity between bars through a steel collar assembly, composed of threaded male and female sleeves. Tensile force is transferred through the steel collar assembly, while compression is directly transferred by bearing between the bars. Mild steel shims are used to fill any gaps between the heads. The grouted-filled sleeve coupler (GC) is composed of a ductile cast iron sleeve in which the spliced bars are inserted and the sleeve is filled with a proprietary high-strength cementitious grout. Tensile and compressive forces are transferred by the deformed ribs on the reinforcing bars into the high-strength grout and then to the cast-iron sleeve.

The HC device is Caltrans prequalified as “Ultimate” splice for No. 4 [D13] through No. 14 [D39] bars, and the GC device is prequalified as “Service” splice for No. 4 [D13] through No. 18 [D57] bars. As noted in Table 1, both Ultimate and Service splices have restrictions on where they can be placed within a structural member. An Ultimate splice may be used in an element expected to undergo large nonlinear deformations (such as a bridge column), whereas a Service splice cannot be used in such an element. Yet, the most important aspect of the placement restrictions is that mechanical splices completely prohibited to be used in plastic hinge zones. Thus, this study has a broader impact on the application of these devices.
3. Experimental Studies

3.1 Half-Scale Column Models

In the first part of the study, five half-scale reinforced concrete bridge column models with circular sections were investigated: one conventional cast-in-place (CIP) benchmark column and four precast columns. The models were identical except for the details in the plastic hinge connection region. The benchmark column was designed using the Caltrans’ Seismic Design Criteria (SDC) (Caltrans, 2010) for a target design displacement ductility of $\mu_c = 7.0$ to achieve large inelastic deformations prior to failure. The geometry and reinforcement details of CIP were selected to be representative of flexural-dominant columns commonly used in California with modern seismic detailing. Table 2 lists the general details for the five half-scale column models.

Table 2: Half-scale column model design parameters

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross-Section</td>
<td>Circular - 24 in [610 mm] Diameter</td>
</tr>
<tr>
<td>Cantilever Height</td>
<td>108 in [2743 mm]</td>
</tr>
<tr>
<td>Longitudinal Reinforcement</td>
<td>11 - No. 8 [D25] Bars</td>
</tr>
<tr>
<td>Longitudinal Reinforcement Ratio</td>
<td>1.92%</td>
</tr>
<tr>
<td>Transverse Reinforcement</td>
<td>No. 3 [D9.5] Spiral - 2-in [51-mm] Pitch</td>
</tr>
<tr>
<td>Transverse Reinforcement Ratio</td>
<td>1.05%</td>
</tr>
<tr>
<td>Aspect Ratio</td>
<td>4.5</td>
</tr>
<tr>
<td>Maximum Clear Cover</td>
<td>1.75 in [44.5 mm]</td>
</tr>
<tr>
<td>Design Axial Load</td>
<td>226 kip [1005 kN]</td>
</tr>
</tbody>
</table>

The remaining four models were precast and utilized hollow concrete shells that contained the same longitudinal and transverse reinforcement as CIP. The hollow shell design would allow for reduced weight during transportation and erection of the column. Once the precast column was installed, the core was filled with self-consolidating concrete (SCC). The connection of the precast column shell to the footing was achieved by using the mechanical reinforcing bar splice described in the previous section. A different connection detail was developed for each mechanical splice, and two column models were tested for each detail: one...
where the connection was made directly to the footing and the second where the column was mounted on a precast pedestal one-half column diameter, $D$, in height (12-in [305-mm]), which was used to reduce the moment demand over the connection location. Longitudinal reinforcing bar passed though the pedestals via grout-filled corrugated steel ducts. Column models were denoted by the type of coupler (“HC” for the up-set headed coupler and “GC” for the grout-filled sleeve coupler) and whether the model included a pedestal (“NP” for no pedestal and “PP” for precast pedestal). Connection details for HCNP, GCNP, and GCPP are shown in Fig. 3. HCPP had the same connection detail as HCNP, but was connected atop a precast pedestal like that shown for GCPP.

Tests were conducted at the Large-scale Structures Laboratory at the University of Nevada, Reno using a single cantilever loading configuration with a servo-hydraulic actuator for lateral loading. Column models were subjected to slow cyclic loading using a drift-based displacement-control loading protocol. Two full push and pull cycles were completed at drift levels of 0.25, 0.5, 0.75, 1, 2, 3, 4, 5, 6, 8, and 10% or until failure, defined to be a significant drop in lateral. A nominally constant axial load of 200 kip [890 kN] was applied to each column model using two hydraulic rams and a spreader beam.

3.1.1 Key Results
In general, the precast models behaved similar to CIP with respect to key response parameters such a force-displacement relationships and energy dissipation. However there were some differences related to formation of plastic hinge mechanisms and displacement ductility capacity.

The measured force-displacement relationships for the precast models HCNP and GCNP are plotted along with that of CIP in Fig. 4. CIP exhibited wide loops, stable post-yield regions, and minimal strength degradation, as expected form a column with modern seismic detailing. The measured response of HCNP was approximately the same as that of CIP except for slight differences in peak load per drift level. The first abrupt drop in lateral load occurred during the second cycle of -10% drift in both CIP and HCNP. The measured response of GCNP was also very similar to that of CIP. However, the first abrupt drop in lateral load occurred during the second cycle of -6% drift for GCNP compared to -10% for CIP. Models with precast pedestals exhibited similar behavior to the counterparts without pedestals. The primary difference being that both HCPP and GCPP failed during the second cycles of +10% and +6% drift, respectively, while the NP counterparts were able to endure an extra half cycle.
The average force-displacement envelope for each precast column was similar to that of CIP (Fig. 5). The curves represent the average envelope from the first cycle of the push-pull loadings. In all cases, the envelope curves for the precast models were the same as CIP up to first yield of longitudinal reinforcing steel. The post-yielding branches for HC models were approximately the same as CIP. The post-yielding branch of GCNP occurred at a slightly higher lateral load compared with CIP due to added stiffness of the grouted coupler connection region. Lastly, the behavior of GCPP was similar to HCPP due to presence of the precast pedestal, but had reduced drift capacity.

Figure 5 also indicates the displacement ductility of each column. CIP and the HC models achieved ductility within 0.5 of the target design ductility of 7.0. The GC models both failed at displacement ductility 4.5. Although this was 35% lower than the target design ductility it may be sufficient in regions with moderate or high seismicity.

The progression of damage in the precast models was similar to that of CIP. The progression of damage for each model is depicted on the average envelope curves (Fig. 5) in terms the damage states defined by Vosooghi and Saiidi (2010), which where: presence of flexural cracks (DS-1), first spall and development of shear cracks (DS-2), extensive cracking and spalling of concrete (DS-3), visible longitudinal and/or transverse reinforcement (DS-4), and on-set of confined concrete core damage (DS-5: imminent failure). Figure 6 illustrates each damage state as observed in CIP. The HC models reached all five damages, which occurred at approximately the same point in the force-displacement history as CIP. The progression of damage for the GC models was similar as CIP except for DS-3 in GCNP, which occurred at a later drift ratio. However, the GC models did not achieve DS-5 prior to failure due to regions of localized damage, which resulted in failure.
One critical difference between CIP and some of the precast models was the formation of the plastic hinge mechanism. As expected, CIP exhibited well-distributed plasticity in the plastic hinge zone. This was also the case for HCNP. On the other hand, GCNP and models with precast pedestals exhibited locations of concentrated plastic rotation, which ultimately resulted in failure. Specifically, these concentrated deformations occurred at the precast column-footing (as noted earlier) and precast pedestal interfaces.

Specifically, all five models exhibited cracking and delamination of concrete at the footing surface due to strain penetration of the longitudinal bars. Yet, the extent of this damage was much greater in the GC models compared to CIP and the HC models. By 6% drift, severe damage to the footing was observed in both GCNP and GCPP (Fig. 7a), which included extensive cracking and delamination of concrete at the footing surface. The loss of the surrounding concrete in the footing resulted in buckling of longitudinal bars and their eventual fracture. This level of damage in the footing was not observed the HC models or CIP until 8.0% drift. Similar to GCPP, delamination of footing concrete in HCPP also resulted in bar buckling and eventual fracture, but did not occur until 10% drift. In the cases of GCNP and the models with pedestals, longitudinal bar fracture occurred approximately 4 in [102 mm] below the footing.
surface due to strain concentrations. Figure 7b shows representative photos of bars removed from HCPP and GCNP. Fracture locations are indicated with an arrow.

In order to assess possible damage, mechanical splices were removed from HCNP and GCNP because they were subjected to the highest moment demand and inelastic deformations. The upset-headed splice did not display any indication of distress or damage. The grouted-sleeves removed from GCNP did not exhibit any damage and the bond between the high-strength grout and reinforcing bar was sound. There was, however, evidence of strain penetration into the coupler sleeve as shallow grout-cone pull-out was observed at both ends.

It was determined that the presence of mechanical splices can influence the formation and behavior a column’s plastic hinge. Figure 8 compares the plastic hinge mechanisms that were observed from half-scale column models.

![Figure 7 Damage due to concentrated deformations](image)

![Figure 8 Plastic hinge mechanisms](image)

3.2 Testing of Individual Mechanically-Spliced Bars

The second portion of the study consisted of 29 uniaxial tests on individual HC and GC devices. The objective was to characterize the component behavior of each splice type under static and dynamic loading. Results also aided the development of analytical models for the half-scale columns. Samples were constructed using No. 8 [D25] ASTM A706 or A615 Grade 60 reinforcing bars for HC and GC samples, respectively. Tests included monotonic static and dynamic tension, single- and multi-cycle elastic slip, and slow cyclic loading. Dynamic tests
were conducted to achieve strain rates (between 50,000 and 100,000 microstrain/sec) similar to those that would be expected during a moderate-to-severe earthquake, and cyclic tests subjected the samples to a single cycle of tension which was increased following application of a compressive stress of 3 ksi [21 MPa].

Both coupler types exhibited consistent results in static and dynamic tests; representative results for monotonic static and dynamic tensile tests are shown in Fig. 9a and b for HC and GC devices, respectively. All samples failed by reinforcing bar fracture, which occurred away from the mechanical splice. Furthermore, both devices were able to sustain increased demand caused by the strain rate effect of dynamic loading without adverse effect to failure locations, measured strains, coupler region behavior, and ductility.

Stress-strain relationships (Fig. 9a and b) indicated that the region incorporating the coupler device had reduced ductility compared to the reinforcing bar. Figure 9c shows representative strain-ratio plots, which depict the relationship between strain over the coupler region and strain in the reinforcing bar. This plot indicates that once strain-hardening begins, which occurred between 100,000 and 150,000 microstrain, the coupler region consistently exhibits reduced deformation compared to the reinforcing bar. For example, the coupler region of GC samples only achieves approximately one-third the strain of the reinforcing bar after the on-set of strain-hardening. It was observed in the half-scale column tests that GC models experienced reduced plastic rotations within the region where grouted couplers where present; whereas, HC columns did not exhibit such behavior. The length of the GC and HC device were approximately 14.5 and 3.5 bar-diameters in length, respectively. Thus, reduced deformation capacity becomes more critical as the length of the splice increases, and longer splices will have a greater effect on the plastic hinge behavior the column.

Cyclic loading tests were used to quantify the behavior of coupler devices upon load reversal. During column tests, HC models exhibited a slight pinch in the force-displacement curve during the cycles returning from peak displacement. It was determined that pinching was caused by permanent deformation or “gap-opening” occurring between the deformed heads of the spliced reinforcing bars. The cyclic loading tests on HC samples quantified the gap opening and closing behavior during load reversals, and it was determined that this behavior accumulated linearly with stress in the bars; the effect of gap-opening was minimal on the energy dissipation of the HC column models. Both devices exhibited results that were comparable with those from monotonic static tests in terms of failure mode and behavior. However, the cyclic tests conducted in this study were limited and did not address large strain reversals, which is an important aspect of understanding the behavior of these devices under seismic loading and should be studied further.

Static Tensile Loading – Solid Lines; Dynamic Tensile Loading – Dashed Lines
4. Analytical Studies

The third part of the study was focused on analytical modeling of the newly developed precast column-footing connections. First, individual component models of reinforcing bars spliced with HC and GC devices where developed. The proposed modeling methods and material models for component models were validated using experimental results, and then were used to develop analytical models of the five half-scale columns. Half-scale columns were modeled with OpenSEES using distributed plasticity frame-elements with uniaxial fiber-sections. These models incorporated the effects of bond-slip rotation at various locations, depending on column type, and predicted longitudinal bar fracture due to low-cycle fatigue (LCF) using the Coffin-Manson LCF model and a linear damage accumulation model. Analytical results were compared with experimental results from half-scale column tests to validate the analytical models.

As an example, Fig. 10 shows the details of the analytical model for the GCNP half-scale column. Analytical models for the remaining four half-scale columns were similar but included different elements or details depending on connection type. The constitutive models for unconfined concrete, confined concrete and longitudinal reinforcing steel were selected based on currently available models in OpenSEES. Single uni-axial fiber section elements were developed for sections that included grouted couplers or precast pedestals with the intent to better capture local behavior and global response of columns. Uni-axial material models for grouted couplers and grout-filled steel duct (within the precast pedestal) were also developed.

Figure 9 Representative results from static and dynamic tensile testing of individual coupler devices
Figure 10 Details of the analytical model for GCNP

Figure 11 shows the global response of the GCNP analytical model compared with corresponding experimental results. In general, there was good correlation between the calculated and measured force-displacement curves with regard to the shape of the loops and the loads at each drift level. On average, the calculated load was 7% higher than the measured load. Figure 7b compares the average and measured envelope curves for GCNP. The curves coincide up to approximately 0.75% drift, at which point the measured envelope begins to soften, while the calculated envelope does not begin to soften until 1.0% drift. Figure 7c shows a comparison between the measured and calculated energy dissipation for GCNP. The calculated energy dissipation was higher than the measured data due to slightly wider hysteresis loops. After yielding of steel, there was approximately 20% difference between the calculated and measured results. Nevertheless, there was good correlation between the measured and calculated results. Similarly, the calculated local behavior i.e. strains and rotations also exhibited good correlation with experimental results.

Analytical models for the other three precast models also had good correlation between the calculated and measured response at global and local levels. The average force-displacement envelopes for these models are shown in Fig. 12 along with the corresponding measured force-displacement envelopes.
Once good correlation between experimental and calculated results was achieved, prototype-scale analytical models were developed to conduct a parametric study and develop design recommendations. The parametric study had two main focuses: 1) sensitivity of GC-type precast columns to changes in critical design parameters, and 2) investigation of the design details for the pedestal used to shift the connection region and reduce moment demand over the mechanical splices.

The half-scale column, GCNP, was designed using an emulative approach and had the following design details: design displacement ductility (DD) = 7.0, aspect ratio (AR) = 4.5, and longitudinal reinforcement ratio (RR) ≈ 2%. These parameters were varied to investigate the behavior of GC columns with different design details. The following parameter values were investigated: DD = 7, 6, and 5; AR = 6; and RR ≈ 1.0%. For each parameter value, a conventional column was designed and the design parameters were used for a corresponding GC column model. This was the same emulative approach that was used to design GCNP.

There were a number of trends identified. Columns with GC connections consistently exhibited higher lateral load capacities than their conventional counterparts due to the added stiffness of the section containing grouted couplers. Despite the difference in capacity, the initial stiffness of conventional and GC columns were approximately the same. A comparison between the elasto-plastic characteristics of the conventional and GN models is shown in Fig. 13. The results for columns with grouted coupler connections with no pedestal (denoted “GN”) and conventional models are plotted on the x- and y-axis, respectively. Thus, if a data point lies to the right of the dashed equivalence line it indicates that the value of the parameter was greater for the GN model compared with the corresponding conventional models and vice versa. The statement above regarding consistently higher lateral load capacity can be identified in Fig. 13a, which depicts the plastic shear force. On the other hand, such a distinct trend cannot be observed in regard to the ultimate displacement and displacement ductility capacity; the results are similar nevertheless. That is, in some cases the conventional column would reach failure prior to the GC column and vice versa. This implies, along with the fact that GC columns consistently have higher lateral load capacities, that emulative design and analysis approaches are not suitable for predicting the behavior of column with grouted coupler connections.
The results from experimental testing of HCPP and GCPP indicated that the pedestal had a significant role in the behavior of the columns. It was found that rotations within the pedestal were relatively small, forcing much of the plastic deformation to the pedestal-footing and column-pedestal interfaces. Thus, a parametric study was conducted to investigate the effect of pedestal height and detailing on the performance of precast columns. Pedestal heights of one-half column diameter, $0.5D$, and a one-full column diameter, $1.0D$, were studied. For each height, a precast (PC) and a cast-in-place (CIP) detail were investigated. Figure 14 identifies the different configurations studies and the associated nomenclature. Both HC and GC connection details were investigated for each configuration.

The parametric study of pedestal details identified that, similar to columns with grouted coupler connections at the column-footing interface, the use of a PC pedestal that incorporates grout-filled steel ducts increases the lateral load capacity of the columns by 5-6%. This can be observed in the force-displacement envelopes shown in Fig. 15. Furthermore, the resulting displacement ductilities were typically lower than a corresponding conventional column. On the other hand, the use of a CIP pedestal can result in the same approximate lateral load capacity, force-displacement relationships, and displacement ductility as a conventional column. For the most part, the height of the pedestal had little effect on the global response of the columns, but had greatest effect on the stress-strain demands in the coupler region. Thus, using a taller pedestal would reduce the likelihood of damage in the mechanically-spliced region. Although numerical data indicates that spalling would likely occur when using a $0.5D$ CIP pedestal, localization of damage in an actual column may prevent damage from progressing above the
pedestal. This would especially be the case for columns with grouted coupler connections due to increased rigidity in the coupler region. Lastly, using a pedestal greater than 1.0D in height may be impractical, and results from parametric study suggest that it would not provide further enhancement to the performance of the column. A summary of results from the parametric study on pedestal details is shown in Table 3, and corresponding parameters that define the elastoplastic curve are shown in Fig. 16.

![Figure 15 Force-displacement results from GC columns with pedestal](image1)

![Figure 16 Key parameters that define the elastoplastic curve](image2)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Repose of a conventional column model with the same details</th>
<th>% difference relative to response of the conventional model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement Ductility</td>
<td>6.52</td>
<td>-3.8  -1.3  -8.5  1.3  -13.5  -5.3  -14.4  -0.9</td>
</tr>
<tr>
<td>Maximum Moment</td>
<td>61604 kip-in 6955 kN-m</td>
<td>6.0  0.1  6.2  -0.2  5.3  0.0  5.6  -0.2</td>
</tr>
<tr>
<td>Maximum Base Shear</td>
<td>285 kips 1269 kN</td>
<td>6.0  0.1  6.2  -0.2  5.3  0.0  5.6  -0.2</td>
</tr>
<tr>
<td>Effective Yield Displacement</td>
<td>2.4 in 62 mm</td>
<td>12.1  0.2  9.2  1.8  14.6  4.7  13.3  4.7</td>
</tr>
<tr>
<td>Ultimate Displacement</td>
<td>16 in 403 mm</td>
<td>7.8  -1.1  0.0  3.1  -0.8  -0.8  -3.0  3.8</td>
</tr>
<tr>
<td>Vp</td>
<td>264 kip 1174 kN-m</td>
<td>6.4  1.1  6.4  1.3  5.6  1.0  6.0  1.0</td>
</tr>
<tr>
<td>Mg</td>
<td>57016 kip-in 6437 kN-m</td>
<td>6.4  1.1  6.4  1.3  5.6  1.0  6.0  1.0</td>
</tr>
<tr>
<td>Effective Stiffness</td>
<td>108 kip/in 19 kN/mm</td>
<td>-5.1  0.8  -2.6  -0.5  -7.9  -3.5  -6.4  -3.6</td>
</tr>
</tbody>
</table>

5. Key Observations

5.1 Half-Scale Column Model Tests

1) Under drift ratios of 6% or less, all four precast models exhibited similar force-displacement relationships, energy dissipation, and damage progression as CIP.

2) The presence of grouted couplers in GCNP resulted in concentrated plastic hinging mechanisms at the column-footing interface. Once delamination of footing concrete occurred bars began to buckle and subsequent fracture occurred.

3) The plastic hinge mechanism for HCNP was essentially the same as that of CIP. Both experienced well-distributed plastic deformation within the first column diameter above
the footing.

4) The primary failure mode in all the columns was fracture of the longitudinal bars. The bars fractured above the footing surface in CIP and HCNP, and approximately 4-5 in [102-127 mm] below the footing surface in the GC models and HCPP due to concentrated plastic rotations.

5) The precast column elements employing GC connections required significantly less installation time than those employing HC connections. Grouted couplers had higher construction tolerances and field dowels that protruded from the footing/ pedestal allowed for easier placement of columns. The transition bar used between headed coupler required tight tolerance, more construction time, and adjustments during installation of the precast columns to the footings.

6) Pedestals were intended to reduce the moment demand over the coupler region and improve ductility. However, no improvement in the drift or displacement ductility capacity was observed. The grout-filled corrugated steel ducts in the pedestal increased section rigidity causing plastic rotations to occur predominately at the column-pedestal and pedestal-footing joints. In the CIP model, the maximum strains occurred within the first one-half column diameter from the footing surface, which is expected. Whereas, pedestals shifted the maximum transverse reinforcement strain to the region above the pedestal.

5.2 Tests on Individual Mechanical Reinforcing Bar Splices
1) Tensile tests of individual couplers indicated that regardless of the loading type, all samples failed due to bar fracture, which occurred away from the coupler itself. Furthermore, there was no apparent damage to the couplers themselves in any of the tests.

2) Both coupler types were able to sustain increased demand caused by the strain-rate effect of dynamic loading without adverse effect to failure locations, measured strains, coupler region behavior, and ductility.

3) Both coupler types exhibited reduced overall ductility in the coupler region compared with the reinforcing bars. After strain in the reinforcing bar reached 20,000 µε, the average strains measured over the coupler regions for HC and GC samples were between 67-100% and 33-50% that of the reinforcing bar up to failure.

4) GC devices exhibited a small grout-cone pull out at the ends of the grouted sleeve. Strain penetration into the sleeve ends formed an unsupported compression strut, which resulted in a shallow wedge of grout pulling out from each end of the coupler sleeve.

5.2 Analytical Studies
1) The analytical models led to similar force-displacement relations compared with test results along with, in most cases, good correlation between the calculated and measured local behavior i.e. strains and rotations.
2) The Coffin-Manson low-cycle fatigue fracture model resulted in reasonable estimate of longitudinal bar fracture for CIP, HCNP, GCNP, and HCPP.

3) The single-element pedestal model exhibited good correlation with global test results despite underestimating strains near the pedestal-column joint.

4) The parametric studies showed that PC pedestals result in increases in lateral load capacity but lower displacement ductilities compared with corresponding conventional columns. CIP pedestals with the height between $0.5D$ and $1.0D$ ($D =$ column diameter) result in force-displacement behavior that is approximately the same as that of a conventional column.

5) The bi-linear constitutive model proposed for the ductile cast-iron material that composed the grouted coupler sleeve provided a reasonable approximation of the actual behavior of the sleeve assembly. Furthermore, a similar statement can be made regarding the equivalent materials properties used to define the behavior of the grouted coupler as a single element with uniform material properties.

6) The modified uniform bond strength equation for bars in grouted ducts proposed by Ou et al. (2010) leads to reasonable estimate of bond-slip behavior due to strain penetration in bars anchored in grouted couplers.

7) A corresponding conventional column model can approximate the global force-displacement behavior of columns with GC connections. However, the conventional model cannot predict ultimate lateral load capacity, which is typically 6-12% larger in the GC column, and does not result in a reliable approximation of displacement ductility capacity or local behavior.

8) After yielding, rotational deformations over the GC connection region are typically 30-40% of that in a corresponding conventional column. Although the rotation is relatively small, the coupler region still accounts for 15-25% of the post-yield top deflection of the column.

6. Conclusions

1) Mechanical bar splices are a viable option for use in ABC substructures in seismic zones, because they can be effective for rapid construction and require detailing that is similar to conventional cast-in-place column.

2) The test and analytical results of this study have shown that the existing provisions in the Caltrans and AASHTO bridge seismic design documents disallowing the use of couplers in plastic hinges are not warranted.

3) Although test results indicated a lower drift capacity in columns with embedded grouted couplers (GC) compared to that of the CIP column, with a drift capacity of 6% the seismic performance of such columns is acceptable.
4) Headed reinforcement coupler connections (HC) fully emulate the response of standard CIP construction in essentially all aspects of the seismic performance. However, these couplers require tight construction tolerances and longer construction time compared with GC couplers.

5) The initial design parameters and reinforcement details for precast columns with mechanical-spliced can be reasonably determined using moment-curvature analysis and lumped-plasticity models.

6) The behavior of precast columns with mechanically spliced connections can be approximated using an analytical model for a corresponding conventional column. However, depending on the length of the splice and relative stiffness to reinforcing bars, an analytical model for a corresponding conventional column may not be able to reliably approximate displacement ductility capacity or localized plastic deformations.

7) Mechanical splices used within a plastic hinge zone can alter the plastic hinge mechanism. Shorter splices, less than 4 bar diameters, will not have a significant effect on the distribution of plasticity whereas larger splices (greater than 14 bar diameters), will have an effect plastic hinge formation and behavior depending of the relative stiffness of the splice.

8) The use of a pedestal can be effective in reducing the demand over the connection region, and can be used to achieve similar performance to a conventional column. However, the effectiveness of the pedestal depends on its height and detailing.

9) Strain concentrations and localized deformation at the column-footing interface can be reduced in GC columns using modified detailing, which employ larger footing dowels. However, this method will result in increased post-yielding stiffness that must be taken under consideration in design.

10) Current code provisions for performance evaluation and acceptance of mechanical splices need to be expanded to reflect cyclic behavior under earthquake loading.