PRECAST COLUMN CONNECTIONS FOR ACCELERATED BRIDGE CONSTRUCTION IN HIGH SEISMIC REGIONS

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ABSTRACT

Numerous transportation agencies have programs to promote the use of accelerated bridge construction (ABC) due to its many advantages such as reducing traffic impact and improving total project delivery time. For non-seismic areas, prefabricated bridge elements have been utilized for years. However, in high seismic areas, connections pose a challenge to designers and may limit the applicability of ABC techniques. In this study, seven half-scale reinforced concrete bridge columns were designed based on Caltrans Seismic Design Criteria and tested under reversed cyclic loading at the University of Nevada Reno. Six different column-to-footing connections were evaluated with respect to a conventional cast-in-place column. Of the six precast models, five employed mechanical reinforcing bar splices within the plastic hinge zone to make the connection with the footing. The connection of the sixth column was made by extending reinforcing bars into corrugated steel ducts located in the footing which were filled with ultra-high performance concrete. The highlights of the study and performance of the ABC connections are presented in this paper. Based on the test results, recommendations are made regarding the design and detailing of ABC connections.

Keywords: Column-to-Footing Connection, High Seismic Zones, Mechanical Reinforcing Bar Splice.

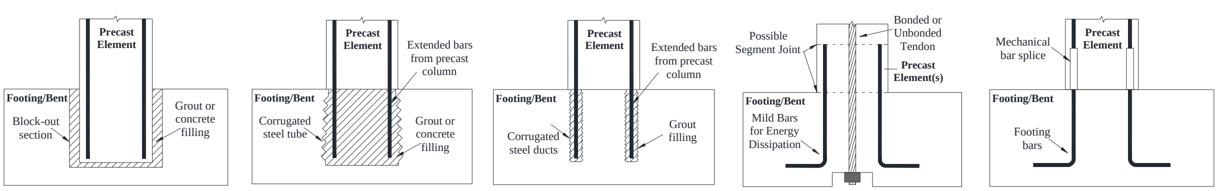
INTRODUCTION

Accelerated bridge construction (ABC) is a collection of design and construction techniques used to expedite bridge construction. In the majority of ABC projects, prefabricated elements are essential to meet strict project time constraints. Along with expediting construction, ABC has been shown to reduce project cost, improve material and element quality, and reduce safety concerns for highway workers[1]. Because of its many benefits, United States federal and state transportation agencies have developed programs to promote ABC.

Precast elements have been widely used in low seismic regions of the country, especially for superstructure members such as deck systems, girders, and barrier rails. In these regions, some projects have also employed prefabricated substructure elements such as bent-caps, footings and abutment components. However, implementation of precast substructural elements has been limited in the high seismic zones due to lack of data on the performance and design of connections details. In order to dissipate energy, substructure connections must be able to undergo large inelastic deformations during a seismic event while transferring the forces to adjacent members.

A number of different ABC substructure connections have been evaluated experimentally and/or analytically for use in moderate and high seismic zones. These connection types included mechanically-spliced connections (i.e. bar couplers), grouted ducts, pocket connections, member socket connections, hybrid connections, integral connections, and connections employing emerging technologies[2]. Some of these connections are shown in Fig. 1. To date, only a few of these connections have been used for column-to-footing joints, most of which have been used for column-to-bent cap joints.

Haber et al.[3] incorporated two types of mechanical reinforcing bar splices (grouted couplers and headed bar couplers) in plastic hinge of four half-scale circular reinforced concrete (RC) bridge columns. The connection details, key findings and performance data from this study are presented in this paper. Grout-filled duct column-to-bent cap connections have been evaluated experimentally and deployed on bridges in Texas[4] and Washington[5]. Experimental results showed that the grouted ducts and pocket connections are emulative of the cast-in-place construction[6,7]. The socket connection was also used in an actual bridge in Washington to connect precast columns to footings[5]. Motaref et al.[8] investigated the seismic performance of a column-to-footing connection with elastomeric rubber plastic hinge which can be categorized as an emerging technology-type connection.



(a) Member socket (b) Pocket-type (c) Grouted ducts (d) Hybrid (e) Bar Splices

Fig. 1 ABC Column-Footing Substructure Connections [Marsh et al.[2]]

This paper presents the seismic performance of seven half-scale RC bridge columns tested under reversed slow cyclic loading until failure. Of the seven columns, one was a conventional cast-in-place column and the remaining six models were precast. Each precast model incorporated a different column-to-footing connection detail within the plastic hinge zone. Connections were specifically designed for use in ABC projects, and employed either mechanical reinforcing bar splices or grout-filled corrugated steel ducts.

COLUMN MODELS DESCRIPTION

The seven half-scale column models, detailed in Table 1, were designed, constructed, and tested at the University of Nevada, Reno (UNR). A benchmark column was first designed according to Caltrans Seismic Design Criteria (SDC) for a target displacement ductility (ultimate displacement / effective yield displacement) of 7.0 such that large inelastic deformations would occur before failure. The geometric and reinforcement details of the benchmark column were to be representative of a typical flexural-dominated bridge column in California. Thus, at half-scale, a 24-*in*. (610-*mm*) diameter cross-section and a cantilever height of 108 *in*. (2743 *mm*) where selected, which corresponded to an aspect ratio of 4.5. The design axial load was 226 *kips* (1005kN), and corresponded to an axial load index (ALI) of 0.10, which is defined as the ratio of axial load to the product of the gross cross-section area and the concrete compressive strength. The benchmark was reinforced longitudinally with 11 - #8 (Ø25*mm*) bars and transversely with a #3 (Ø9.5*mm*) spiral at a 2-*in*. (51-*mm*) pitch. This configuration resulted in longitudinal and transverse reinforcement ratios of 1.9% and 1.0%, respectively. The benchmark column details were used to construct a baseline column model (CIP) used to evaluate the performance of precast models.

Table 1 Test Matrix of Column Models ABC Base Connections

Column ID	Connection Detail	Investigating
CIP	conventional connection	baseline column model
GCNP	grouted couplers w/ no pedestal	column performance with grouted couplers placed above column-footing interface
GCPP	grouted couplers w/ precast pedestal	column performance with grouted couplers placed 12 *in*. (305 *mm*) above column-footing interface
GCDP	grouted couplers w/ debonded bars in CIP pedestal	improving displacement ductility capacity of previously tested columns with grouted couplers
HCNP	headed bar couplers w/ no pedestal	column performance with headed bar couplers placed above column-footing interface
HCPP	headed bar couplers w/ precast pedestal	column performance with headed bar couplers placed 12 *in*. (305 *mm*) above column-footing interface
PNC	reinforcing bars embedded in UHPC-filled ducts	column performance with column-to-footing connection with UHPC

Each of the six precast columns models employed a different plastic hinge connection detail, which are shown in Fig. 2. Three of the six precast models employed grouted sleeve couplers (denoted “GC”), which can be cast within the precast element leaving connection ports for reinforcing bar dowels protruding from an adjacent member. Once the two members are joined, the sleeves are filled with high-strength cementitious grout to complete the connection. In GCNP, the precast column joint was located at the column-footing interface.

To investigate if lower moment demand over coupler location would improve performance, two columns with GC connections were installed atop pedestals. The precast column joint for GCPP was located 12 *in*. (305 *mm*) above the footing surface on a precast pedestal. Longitudinal bars passed through the precast pedestal via corrugated galvanized steel ducts, which were filled with normal-strength cementitious grout prior to installing the precast column. GCDP was similar to GCPP but had a cast-in-place pedestal with longitudinal bar that were debonded from concrete using duct-tape wrapping to improve ductility.

Two precast models employed headed bar couplers, denoted “HC”, to join reinforcing bars within the precast column with those from the footing using a 12-*in* (305-*mm*) transition bar. Prior to placement of the transition bars, a transverse reinforcing steel spiral was placed, which was tied after transition bar placement. Once the transition bars and spiral were installed, the connection was completed by pumping normal-strength cementitious grout into the block-out section. Similar to GCNP, one HC column model (HCNP) was connected directly to the footing. The second HC column model, denoted “HCPP”, employed the same precast pedestal configuration as GCPP.

The last precast column, denoted “PNC”, was connected to the footing by embedding the column longitudinal bars into ultra-high performance concrete (UHPC)-filled corrugated steel ducts located within the footing. UHPC is a cementitious fiber-reinforced concrete with superior durability, ductility, and tensile/compressive strength compared to conventional concrete or cementitious grout[10]. Through a series of reinforcing bar pull-out tests on UHPC-filled ducts, it was determined that the bond strength of UHPC is seven times stronger than the conventional concrete[11]. This had two major implications: 1) reinforcing bar strain penetration into the footing would be significantly less than bars embedded in conventional concrete, and 2) the required anchorage length could be reduced for un-hooked bars compared to those anchored in conventional concrete. Therefore, an 8-*in*. (203-*mm*) length of each longitudinal bar was debonded from concrete at the column-footing interface using duct tape to avoid strain concentrations, which could cause premature bar fracture. All precast columns were hollow prior to installation, and were filled with self-consolidating concrete (SCC) to complete construction. A summary of the measured day-of-test material properties for each column model are provided in Table 2.

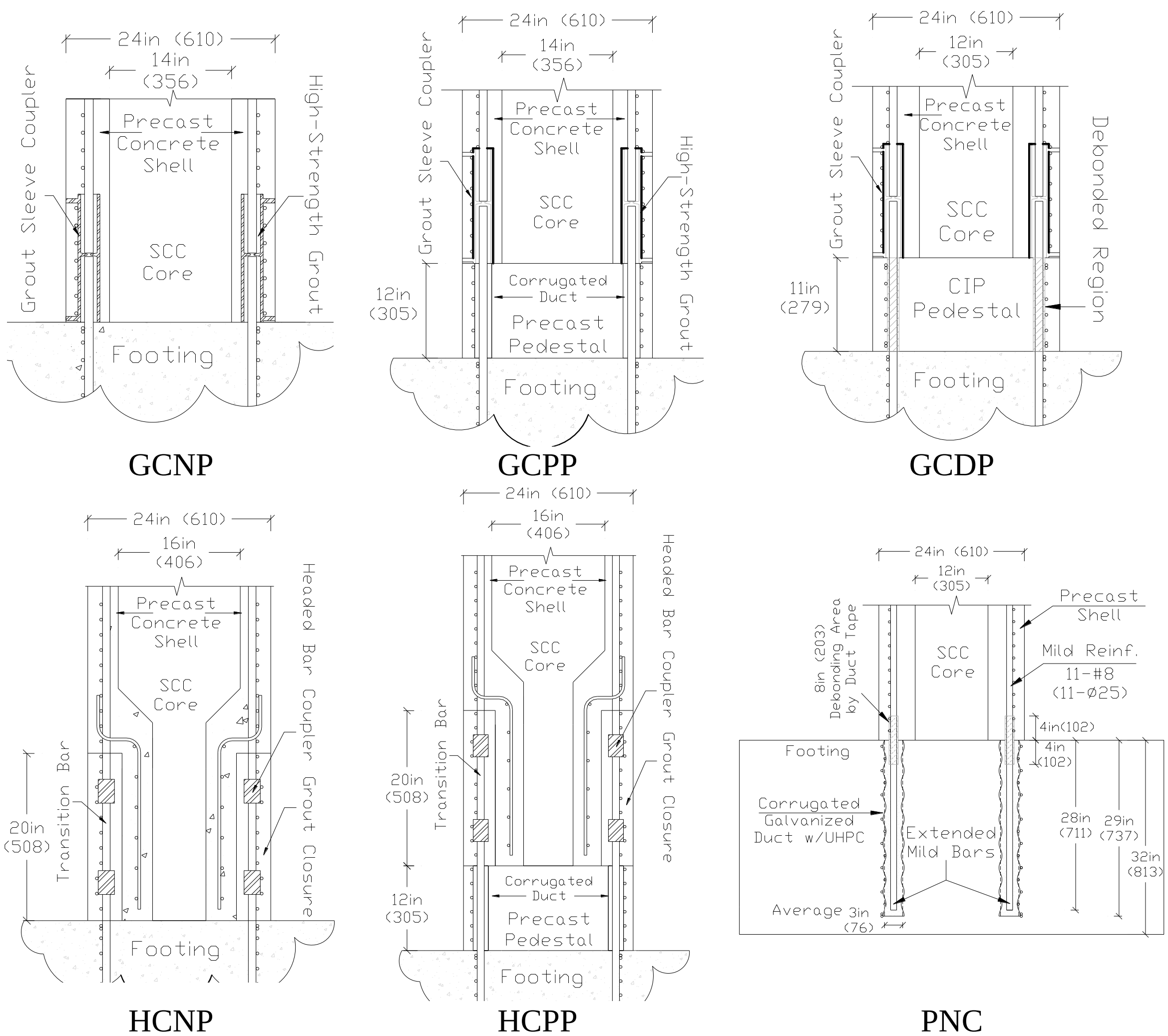


Fig. 2 Precast Column Models ABC Base Connection Details (Unit: *mm*)

Table 2 Average Test Day Strength of Materials, *psi* (*MPa*)

Column Model	Footing	Pedestal	Shell	Core, SCC	Coupler Grout	Pedestal Grout	UHPC	Reinforcing Steel
CIP	5415 (37.3)	N.A.	4445 (30.6)	N.A.	N.A.	N.A.	N.A.	ASTM A615 Gr. 60 Yield Stress = 67 *ksi* (461) Ult. Stress = 111 *ksi* (765)
GCNP	5500 (37.9)	N.A.	4230 (29.1)	5000 (34.5)	16410 (113.1)	N.A.	N.A.	
GCPP	5722 (39.4)	4200 (28.9)	4200 (28.9)	5140 (35.4)	15850 (109.3)	7015 (48.3)	N.A.	
GCDP	5660 (39.0)	3210 (22.1)	3210 (22.1)	8750 (60.3)	16970 (117.0)	N.A.	N.A.	ASTM A706 Gr. 60 Yield Stress = 67 *ksi* (461) Ult. Stress = 92- 95 *ksi* (634 - 65)
HCNP	5645 (38.9)	N.A.	3860 (26.6)	5835 (40.2)	N.A.	N.A.	N.A.	
HCPP	5690 (39.2)	4300 (29.6)	4300 (29.6)	5240 (36.1)	N.A.	7060 (48.7)	N.A.	
PNC	5485 (37.8)	N.A.	3290 (22.7)	9510 (65.6)	N.A.	N.A.	22970 (158.4)	

Each specimen was tested under slow revered cyclic loading until failure using the drift-based protocol shown in Fig. 3. Two full cycles were applied at each drift level until substantial loss of lateral load capacity occurred. In some cases, after loss of capacity, additional cycles of a particular drift level were applied as determined during the test. Columns were loaded with a servo-hydraulic actuator in the single cantilever test setup shown in Fig. 4. A 200-*kip* (890-*kN*) axial load was applied to the columns using two hollow-core rams, and was held nominally constant for the duration of the test using a nitrogen accumulator. During each test, column tip displacement, plastic hinge curvatures, and internal reinforcing bar strains were digitally recorded.

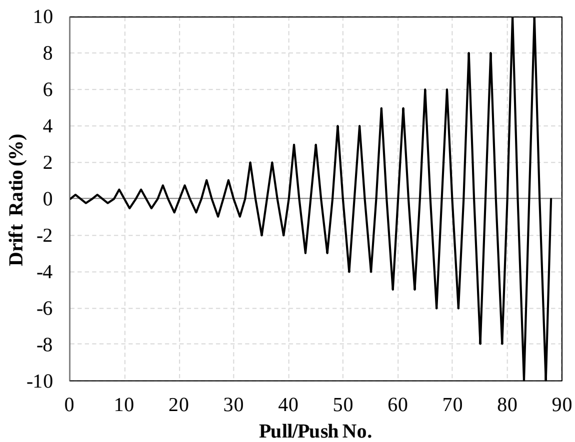


Fig. 3 Drift-Based Loading Protocol


Fig. 4 Column Test Setup

TEST RESULTS

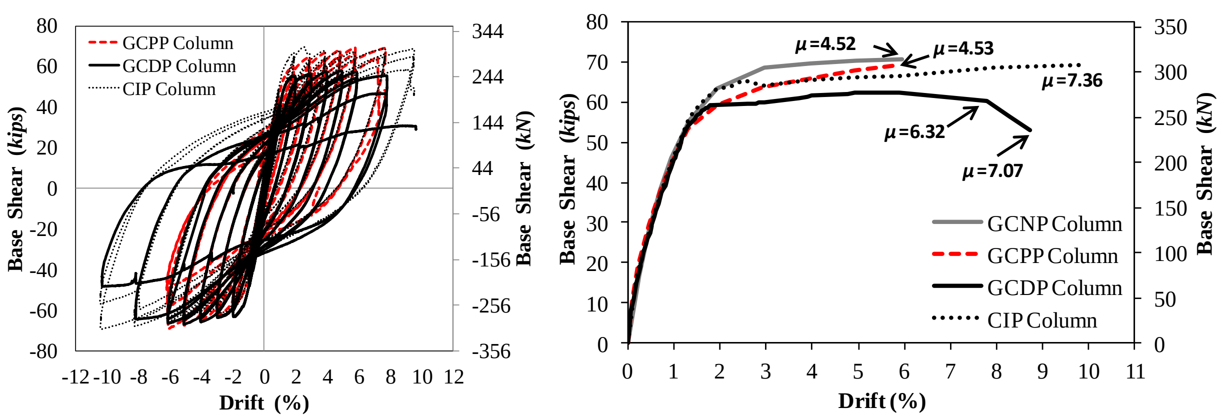
The following sections present the general results from this study including column failure modes, force-displacement relationships, plastic hinge damage, and energy dissipation capacities. More detailed discussion of test results is presented in other publications by the authors[3,11,12].

FAILURE MODES

Failure of reinforced concrete columns can be defined in a number of different ways i.e. initiation of confined core crushing, longitudinal bar buckling or rupture, or transverse bar fracture. In this study, each column model experienced longitudinal bar fracture, which was in some cases initiated by longitudinal bar buckling. Thus, in the context of this paper, a column was considered to have failed once a 15% decrease in lateral load occurred due to longitudinal bar fracture.

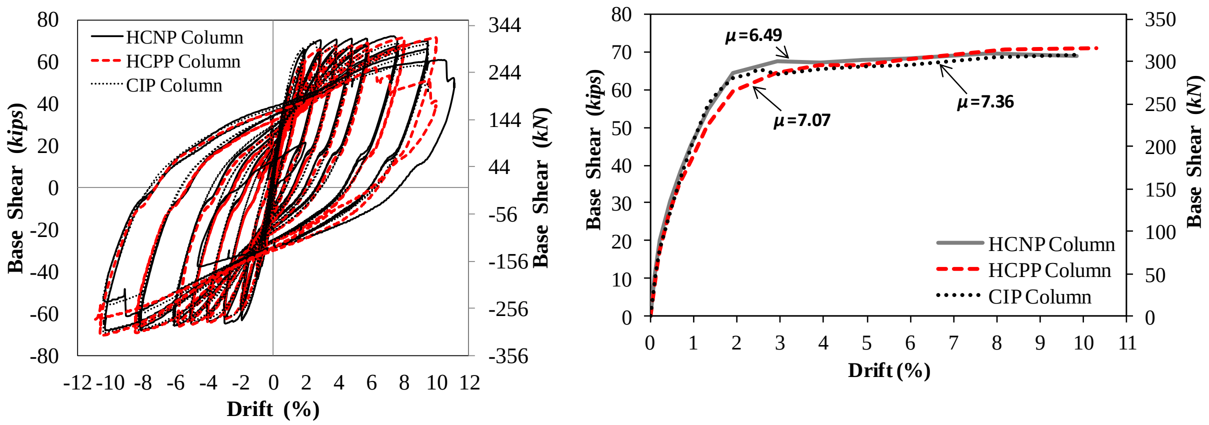
FORCE-DISPLACEMENT RELATIONSHIPS

Force-displacement hysteresis and average envelope curves for the GC-type columns are shown in Fig. 5. The hysteresis of GCNP is not shown for clarity, but closely resembled GCPP. All three GC columns exhibited stable hysteresis loops with minimal strength degradation up to the first bar fracture. Drift capacity was defined as the largest drift level prior to longitudinal bar fracture of loss of 15% lateral load capacity. The drift capacities of GCNP and GCPP were both 6% resulting in displacement ductility capacity of 4.5 for each column (based on first bar fracture cycle criteria). These were substantially lower than the drift and displacement ductility capacities of CIP, which were 10% and 7.3, respectively. Lower displacement capacity of the GC columns was caused by concentrated plastic rotations that occurred at the column-footing interface, which results in premature fracture of longitudinal bars. These columns were stiffer in regions where grouted couplers and grout-filled steel ducts were present, which resulted in significant damage within the footing at large drift levels. In the case of GCDP, which employed debonded bars within a CIP pedestal, the observed damage in the plastic hinge was more similar to CIP than the other GC columns. Analysis of strain data also indicated well-distributed reinforcing bar plasticity along the pedestal height, which resulted in better drift and displacement ductilities capacities; 8% and 6.3 (based on the first bar fracture cycle criteria), respectively. The displacement ductility capacity of the GCDP column would be 7.1 based on 85% ultimate base shear capacity criteria. It should be noted that the lower base shear capacity of GCDP compared with CIP was a result of lower material strengths (Table 2). The force-displacement relationships of the HC models are shown in Fig. 6. Both HC columns exhibited similar performance compared to CIP in terms of cyclic behaviour, base shear and drift capacities (10% drift). The HC models also had similar displacement ductility capacities compared with CIP. The force-displacement relationships for PNC are shown in Fig. 7. The column showed stable hysteresis loops and minimal strength degradation. The drift capacity of PNC was 8% resulting in a displacement ductility capacity of 6.3. The displacement ductility of PNC was slightly lower than CIP due to lower concrete strength, which reduced the column resistance against bar buckling. Caltrans SDC requires that a ductile member must have a design displacement ductility capacity of at least 3.0. All of the proposed ABC connections met this requirement. Recall, the target design displacement ductility of CIP was 7.0; the measured ductility capacity was 7.36. All of the precast models achieved ductility capacities close to the target with the exception of GCNP and GCPP, which were approximately 64% lower than the target value.



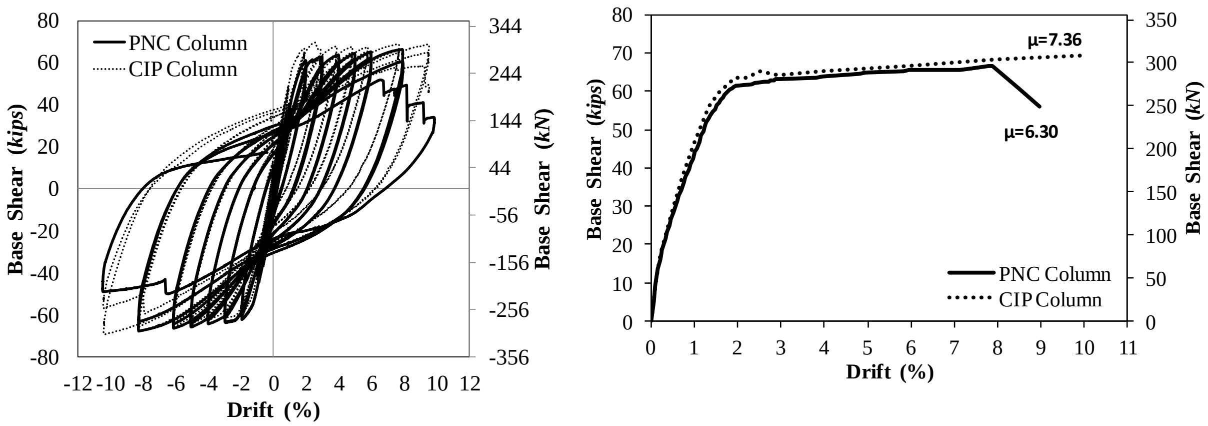
(a) Hysteresis (b) Average Backbone

Fig. 5 Force-Displacement Relationships for Columns with Grouted Sleeve Couplers (GC)



(a) Hysteresis (b) Average Backbone

Fig. 6 Force-Displacement Relationships for Columns with Headed Bar Couplers (HC)



(a) Hysteresis (b) Average Backbone

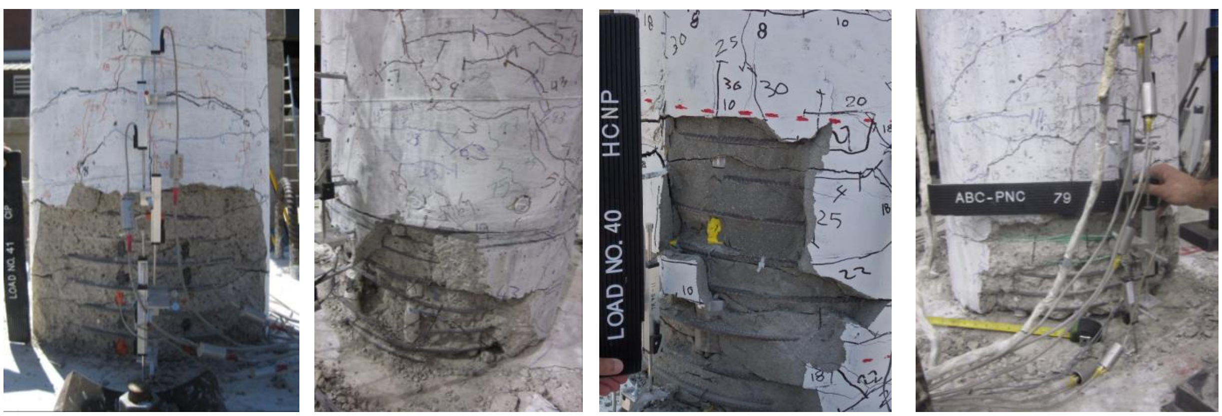
Fig. 7 Force-Displacement Relationships for the Column with UHPC-Filled Duct Connection (PNC)

OBSERVED DAMAGE

Figure 8 shows the plastic hinge damage at 6% drift for CIP, GCPP, GCNP and HCPP. Expensive spalling exposed several transverse bars, which can clearly be observed in all seven columns at 6% drift. Furthermore, longitudinal bars where visible on CIP along with HCNP, GCDP, and PNC. The corrugated steel ducts were visible in both models incorporating a precast pedestal. As discussed in the previous section, GCNP and GCPP both failed prematurely compared with CIP due to concentrated deformation at the column-footing interface, which caused bar fracture a few inches below the footing surface. This is evident in Fig. 8 by the extensive damage at the footing surface in GCNP and GCPP. Fig. 9 shows the plastic hinge damage of CIP, GCDP, HCNP and PNC at 8% drift. These four models exhibited damage penetration to the confined concrete (or grout in the case of HCNP) core and several visible reinforcing bars. Buckled and fractured longitudinal bars were observed in GCDP and PNC at 8% and in CIP, HCNP, or HCPP at 10% drift. In GCDP, damage was localized within the CIP pedestal due to added stiffness provided by the grouted sleeve couplers located above the pedestal. In general, the progression of damage observed in each precast model was similar to that of CIP for most drift levels. Upon completing testing, the connection zones (i.e. couplers or UHPC-filled ducts) of each column were inspected for damage. In all column models, the critical connection mechanism was sound and damage-free. That is, bar pull-out from grouted couplers or UHPC-ducts was not observed. This indicates that the connection details used in this study can withstand numerous cycles of large inelastic deformation. However, the use of couplers within the plastic hinge can potentially alter the formation and behaviour of the plastic hinge.

CIP GCPP GCNP HCPP

Fig. 8 Plastic Hinge Damage of Select Column Models at 6% Drift Ratio



CIP GCDP HCNP PNC

Fig. 9 Plastic Hinge Damage of Select Column Models at 8% Drift Ratio

ENERGY DISSIPATION

Figure 10 shows the cumulative energy dissipation, which is defined as area under the force-displacement hysteresis loops, for each column model as a function of drift. The curve for each column is shown up to the point of failure as defined by previous sections. Since each column had slightly different material properties, the energy dissipation was normalized by the base shear capacity of the column. Each GC column exhibited equal or better energy dissipation compared to the benchmark CIP column. On the other hand, both HC columns showed slightly less energy dissipation than the benchmark CIP column, which was due to a slight pinch in each hysteresis loop. The pinch was caused by gap-opening and closing between the heads of the HC coupler device during loading. Nevertheless, the difference in energy dissipated compared with CIP was insignificant. Similar to the GC columns, the energy dissipation per drift level for PNC was also slightly higher than CIP column. In general, the precast columns had energy dissipation capacities similar to that of CIP for each drift level.

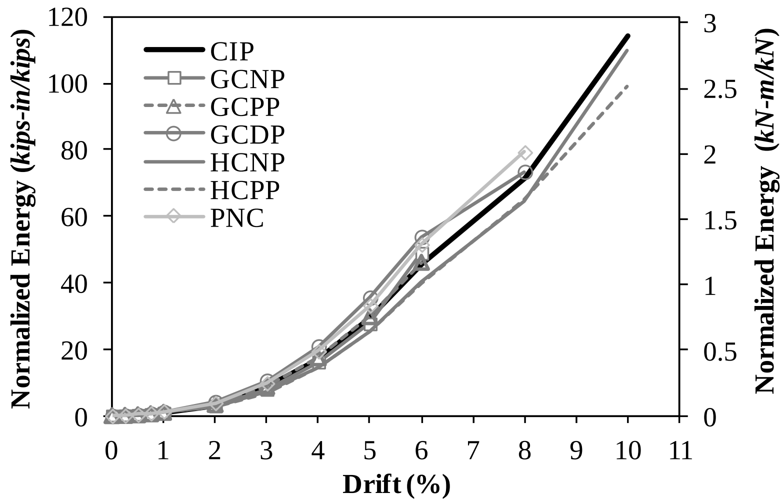


Fig. 10 Energy Dissipation of ABC Columns Normalized to Column Base Shear Capacity

SUMMARY AND CONCLUSIONS

The seismic performance of seven half-scale RC bridge column models was investigated to evaluate applicability of different types of ABC column-to-footing connections for high seismic zones. Six different connection details were tested, and employed either grouted sleeve couplers, headed bar couplers, or a UHPC-filled duct system to join precast columns with cast-in-place footings. In three models, pedestals were used to reduce the moment demand over the precast connection region. The main objective was to determine whether these new connections were emulative of cast-in-place construction. That is, could these new connections be designed using conventional methods to achieve behavior similar to conventional CIP columns. The seventh column model was cast-in-place and served as benchmark. The key findings and recommendations from this study are as follows:

- All precast columns showed similar performance up to 5% drift compared to CIP in terms of hysteresis behavior, base shear capacity, damage progression, and energy dissipation.
- The drift and ductility capacities (based on first fracture) for each model were as follows:
    - CIP – 10% and 7.36
    - GCNP – 6% and 4.52
    - GCPP – 6% and 4.53
    - GCDP – 8% and 6.32

  - HCNP – 10% and 6.49
  - HCPP – 10% and 7.07
  - PNC – 8% and 6.3
- Although mechanical reinforcing bar splices show promise for ABC connections in seismic zones, they may alter the formation and behavior of the column plastic hinge. It is recommended that grouted sleeve couplers be used with a cast-in-place pedestal and unbounded longitudinal bars in pedestal.
- Precast pedestals with grout-filled corrugated steel ducts do not improve drift or displacement ductility capacity and are not recommended.
- Among the different connection types, UHPC-filled ducts and grouted sleeve couplers were easiest to construct and provided the largest tolerances. On the other hand, connections using headed bar couplers required tight tolerances making installation difficult. However, all the proposed connections are sufficiently practical to be utilized in actual bridges.
- Based on the finding of this study, GCDP, HCNP, and PNC are completely emulative of conventional construction. Thus, these connection types are recommended for column-to-footing joints in high seismic regions. However, all of the connection types could be used for ABC connections in regions with lower seismic risk.

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REFERENCES

1. Culmo, M. P. “Accelerated Bridge Construction - Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems,” *Federal Highway Administration Report No. FHWA-HIF-12-013*, 2011, 349 pp.
2. Marsh, M.L., Wernli, M., Garrett, B.E., Stanton, J.F., Eberhard, M.O. and Weinert, M.D. “Application of Accelerated Bridge Construction Connections in Moderate-to-High Seismic Regions,” *National Cooperative Highway Research Program (NCHRP) Report No. 698*, Washington, D.C., 2011, 324 pp.
3. Haber, Z.B., Saiidi, S.M. and Sanders, D.H. “Seismic Performance of Precast Columns with Mechanically Spliced Column-footing Connections,” Accepted for Publication, *ACI Structural Journal* , Vol. XX, 2013, pp xx-xx.

4. Brenes, F.J., Wood, S.L. and Kreger, M.E. "Anchorage Requirements for Grouted Vertical-Duct Connectors in Precast Bent Cap Systems," *FHWA/TX-06/0-4176-1*, Center for Transportation Research, University of Texas at Austin, 2006, 251 pp.
5. Khaleghi, B., Schultz, E., Seguirant, S., Marsh, L., Haraldsson, O., Eberhard, M. and Stanton, J. "Accelerated Bridge Construction in Washington State: From Research to Practice," *PCI Journal* , 2012, pp 34-49.
6. Matsumoto, E.E., Waggoner, M.C., Sumen, G. and Kreger, M.E. (2001). Development of a Precast Bent Cap System. Center for Transportation Research, The University of Texas at Austin: FHWA Report No. FHWA/TX-0-1748-2.
7. Restrepo, J.I., Tobolski, M.J. and Matsumoto, E.E. "Development of a Precast Bent Cap System for Seismic Regions," National Cooperative Highway Research Program (NCHRP) Report 681, Washington, D.C., 2011, 116 pp.
8. Motaref, S., M. Saiidi, and D. Sanders. "Experimental Study of Precast Bridge Columns with Built-In Elastomer," Transportation Research Record: Journal of the Transportation Research Board, Bridge Engineering , Volume 3, 2010, pp 109-116.
9. Caltrans, "Seismic Design Criteria (SDC), version 1.4," Sacramento, CA.: California Department of Transportation, 2006.
10. Graybeal, B. (2010). Behavior of Field-Cast Ultra-High Performance Concrete Bridge Deck Connections Under Cyclic and Static Structural Loading. Federal Highway Administration report No. FHWA-HRT-11-023.
11. Tazarv, M. and Saiidi, M.S. "Emulative Moment-Resistant RC Bridge Column-Footing Connection for Accelerated Bridge Construction in High Seismic Zone," 7th National Seismic Conference on Bridges & Highways, Oakland. Paper No. B6-2, 2013, 10 pp.
12. Haber, Z.B., Saiidi, M.S., and Sanders, D.H., "Precast Column-Footing Connections for Accelerated Bridge Construction in Seismic Zones," Center for Civil Engineering Earthquake Research, Department of Civil and Environmental Engineering, University of Nevada, Reno, Nevada, Report No. CCEER-13-08, April 2013.