# A PRECAST BRIDGE BENT SYSTEM FOR SEISMIC REGIONS

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## Abstract

This paper describes precast concrete bridge bent connections that are suitable for high seismic zones. Lateral load tests on the top connection system have shown that it has strength and ductility similar to those of a comparable cast-in-place connection. Tests on the bottom connection system are ongoing, and a demonstration project will be built later this year.

## Introduction

Bridge construction frequently leads to traffic delays, which incur costs that can be measured in time, money and wasted fuel. Agencies are therefore seeking methods for accelerating bridge construction, referred to as ABC. Use of precast concrete for bridge substructures in seismic regions represents promising technology for ABC (Matsumoto et al., 2001). Precast connections are typically made at the beam-column and column-foundation interfaces to facilitate fabrication and transportation. However, for structures in seismic regions, those interfaces represent the locations of high moments and large inelastic cyclic strain reversals. Devising connections that are not only sufficiently robust to accommodate those cyclic loads, but are also readily constructible, are the primary challenges for ABC in seismic regions. The precast concrete bridge bent system described in this paper is intended to meet those challenges. Different connection systems are used at the column-to-foundation and the cap-beam-to-column interfaces, because the conditions at each location offer unique opportunities. The cap-beam connection has already been tested, and has proved to be very satisfactory. Results from those tests are presented first. The foundation connection test specimens have been constructed and, at the time of writing, are being tested.

## Beam-to-Column Concept

**Concept:** The connection concept consists of bars that project from the column and are grouted into ducts in the cap beam. This concept has been used before by others (Getty, *web site*). Such designs have used a conventional arrangement of longitudinal reinforcement, consisting of 18 #11 bars in a 5-foot-diameter (1.52 meters) column. In such a configuration, the vertical ducts must be small to fit between the horizontal beam

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bars. Then, fitting the column bars into small ducts in the cap beam makes assembly tedious and the system is not readily constructible.

The system developed is shown in Figure 1 and uses a small number of large bars, with correspondingly large ducts, to facilitate fit-up on site. In a typical 5-foot circular column, eight #18 bars in 8-inch-diameter ducts will often suffice. The major obstacle then lies in anchoring the bars in the available space.

The approach taken to solve this problem is to note that in a typical dropped cap beam design a diaphragm is cast in place over the cap beam once the girders have been placed. That diaphragm, shown in Figure 1, provides additional anchorage length for the protruding column bars that can be used together with the anchorage in the ducts to resist the cyclic forces on the bar caused by seismic loading in the transverse direction. The diaphragm also serves to distribute bending under longitudinal loading to stirrups that assist with force transfer. The ducts alone then need to provide only enough development to ensure strength and stability during erection.

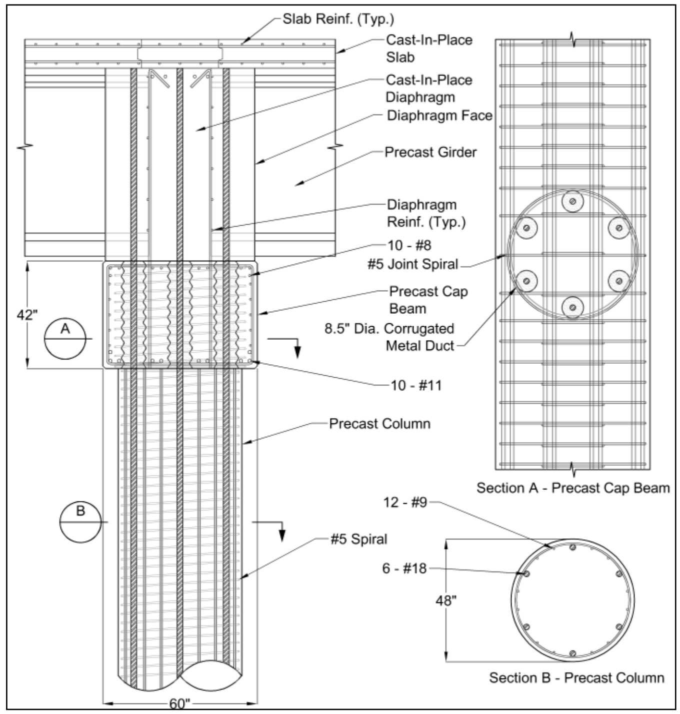



**Figure 1 – Details of full-scale connection**

**Testing:** The anchorage characteristics of large bars were first investigated by conducting full-scale pull-out tests on #10, 14 and 18 bars grouted into corrugated ducts. These tests confirmed that even #18 bars could be fully anchored in a length significantly shorter than the depth of the cap beam (Steuck et al. 2007, 2009). Those tests combined with earlier work (Raynor et al., 2002; Precast, 2004; Moustafa et al., 1989) showed that, under static loading, the lengths needed to provide anchorage for yield and fracture were approximately $6d_b$ and $10d_b$ respectively. An additional allowance of 50 percent is used to account for cyclic loading. The 50 percent is conservative, and is supported by the cyclic tests in Raynor's study. Even with the 50 percent increase in $l_d$ to account for cyclic loading, the bars can be anchored within the depth of the cap beam to achieve fracture rather than pullout.

Four scaled sub-assemblage tests were performed to evaluate the seismic performance of the proposed connection (Pang et al., 2009). The primary study variable was the anchorage of the longitudinal bars. In two specimens, those bars were debonded over a short length near the beam-column interface to reduce the strain concentration that might otherwise occur there, because the bond resistance provided by the grouted duct is very high.

Each specimen was a 40-percent-scale sub-assemblage of the proposed beam-to-column connection. The full-scale prototype was assumed to be a 4-foot-diameter (1.22 meter) circular column. Table 1 and Figure 2 show the details of the scaled test specimens. Specimen REF is a scaled model of a typical Washington State cast-in-place bridge column, with 16 #5 bars evenly distributed round the perimeter, giving a longitudinal reinforcement ratio, $\rho$, of 1.58 percent. The transverse reinforcement consists of 1/4-inch-diameter (6 mm) spiral spaced at 1-1/4-inches (32 mm). This specimen provides a baseline for evaluating the performance of the proposed system.

The other three specimens, LB-FB, LB-D1, and LB-D2, represent possible variations of the proposed precast connection. The columns were reinforced longitudinally with six #8 bars that were grouted into 4-inch-diameter corrugated metal ducts in the cap beam and further anchored in concrete within the diaphragm, providing a longitudinal reinforcement ratio, $\rho$, of 1.51 percent. In Specimen LB-FB the bars were fully bonded into grouted ducts, whereas in specimens D1 and D2, two methods of local debonding were studied. The local debonding was intended to reduce the strain concentration at the beam-column interface that is caused by the short development lengths. The bars in LB-D1 and LB-D2 were debonded over a length of 8 bar diameters, $d_b$, into the cap beam using two different methods. LB-D1 was debonded using a 1-inch-diameter (25 mm) SCH-40 PVC pipe slit longitudinally, fitted tightly around the #8 bar, taped together, and sealed with caulk at the ends. LB-D2 was debonded using a 1-inch-diameter (25 mm) SCH-30 PVC pipe. That pipe fitted more loosely round the bar and was easier to construct, but provided no restraint against buckling. The detail was constructed by sliding the pipe over the bar and sealing it with caulk at the ends.

**Table 1 - Test Matrix**

| **Specimen** | **Description** | **$\rho$ (%)** | **Longitudinal Reinforcement** | **$f'_{co}$ (ksi)** | **$P_{axial}$ (kips)** | **$\alpha_{axial}$ (%)** |
|---|---|---|---|---|---|---|
| REF | Reference cast-in-place reinforced column | 1.58 | 16- #5 | 6.83 | 240 | 11.2 |
| LB-FB | Precast column with bars fully grouted in corrugated ducts in beam | 1.51 | 6- #8 (12- #3 for spacing) | 8.34 | 212 | 8.1 |
| LB-D1 | LB-FB with bars debonded 8 $d_b$ in the grouted ducts using Method 1 | 1.51 | 6- #8 (12- #3 for spacing) | 7.69 | 260 | 10.8 |
| LB-D2 | LB-FB with bars debonded 8 $d_b$ in the grouted ducts using Method 2 | 1.51 | 6- #8 (12- #3 for spacing) | 6.20 | 240 | 12.3 |

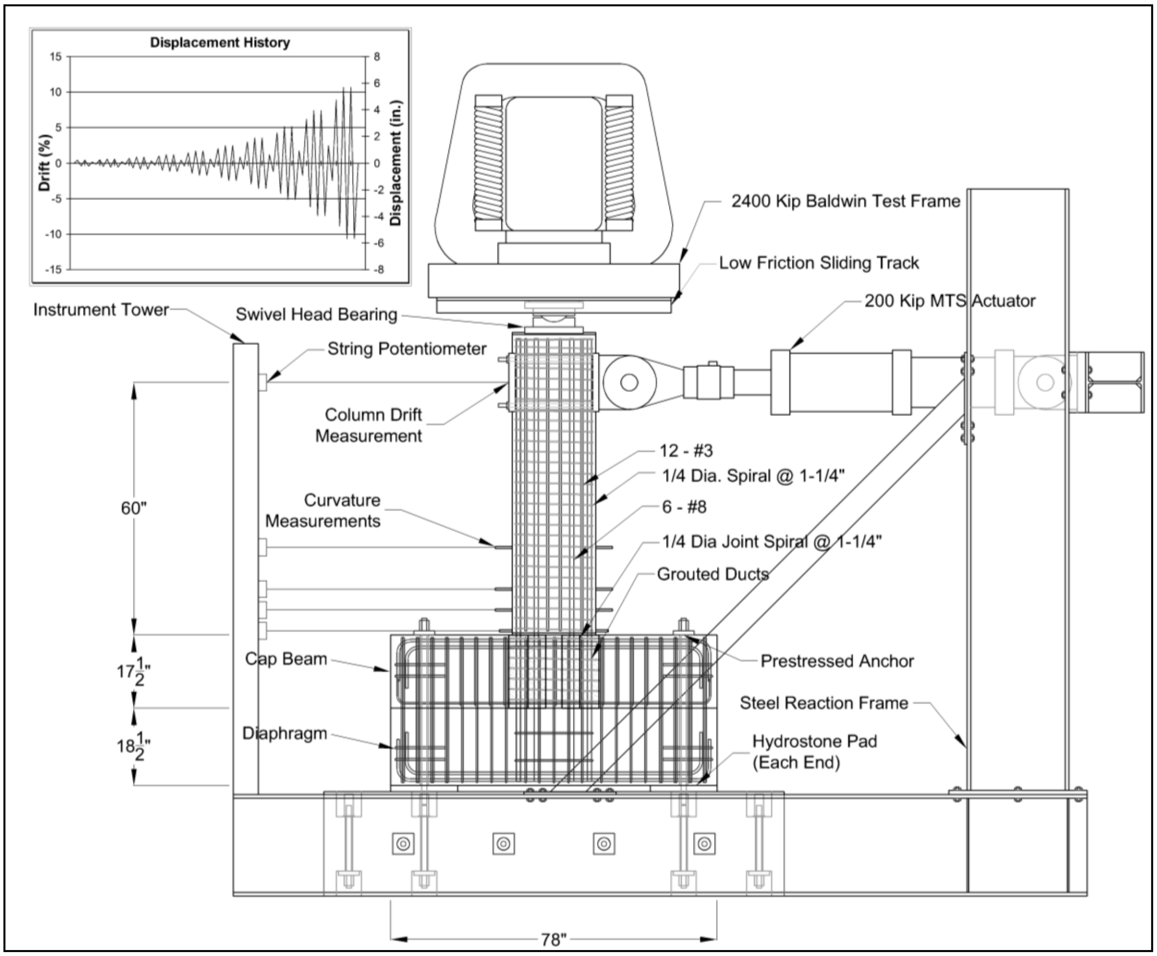



**Figure 2 - Test setup showing precast connection**

Twelve #3 bars were added to each precast column to satisfy AASHTO spacing requirements. They stopped at the interface and so provided no additional flexural capacity. The spiral reinforcement in the columns was the same as in the reference specimen, and it continued at the same spacing into the cap beam to confine the joint region. A thin grout pad was also provided at the beam-column interface to simulate field erection of the precast pieces. Fluid, high-strength, non-shrink grout with an average compressive strength of 9 ksi (62 MPa) at 5 days was used. Grade 60 (400 MPa) bars were used for the mild steel reinforcement, while Grade 90 (600 MPa) wire was used for the spirals. The design concrete strength was 6 ksi (42 MPa).

**Experimental Results:** Each specimen was tested under constant axial load and a cyclic lateral displacement history. The test set up is shown in Figure 2. The selected lateral loading protocol consisted of three cycles at each of a series of increasing displacements.

All four specimens demonstrated nearly identical force-displacement responses and levels of physical damage. Specimens REF, LB-FB, LB-D1, and LB-D2 maintained 80 percent of their peak lateral resistance out to drifts of 5.5, 5.2, 5.7, and 5.8 percent.

Figure 3 shows the load-deflection curves for each specimen. They are remarkably similar, despite the different construction methods. The slightly different peak loads are mainly attributable to minor differences in the applied axial load. There is a small amount of pinching as the system crosses zero displacement, caused by the fact that the compressive force is resisted by the bars alone prior to closing of the cracks in the concrete. There is also little loss in strength until the drift reaches at least 4 percent, despite the increasing damage accumulating in the plastic hinge region.

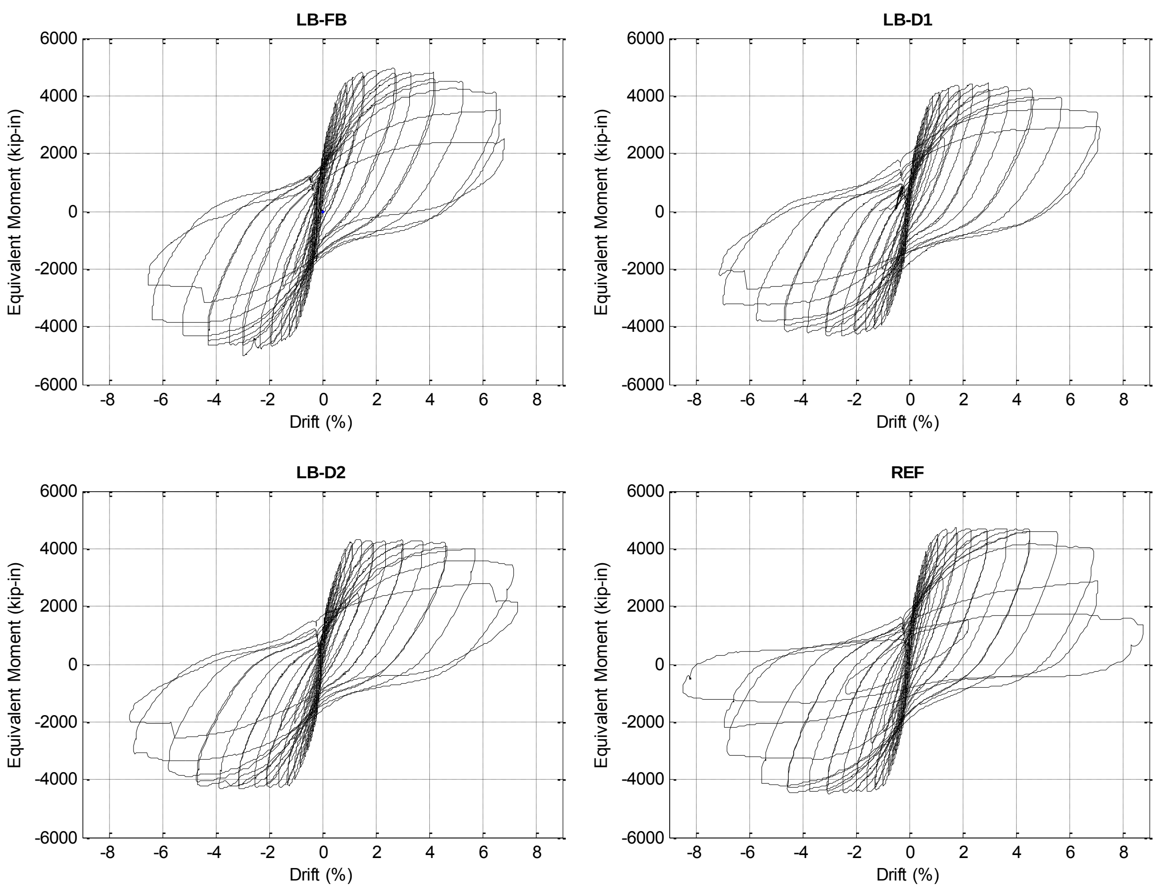



**Figure 3 - Equivalent Moment vs. Drift Plots**

The majority of deformation for specimens LB-FB, LB-D1 and LB-D2 resulted from the opening of a large localized crack at the interface. Rotations measured over the bottom 1.5 inches (38 mm) of the column accounted for more than 90 percent of the total column displacement. This approach, in which the members behave as essentially rigid bodies, while the connections deform, has been successfully used in precast building design and tested in the PRESSS program (Nakaki et al., 1999). In contrast, in Specimen REF, the curvature was more evenly distributed over the bottom 3.5 feet (1.07 meters) of the column, as is common in cast-in-place systems. In it, the crack width at the interface was about the same as the sum of the flexural cracks above.

The types and amount of physical damage were nearly identical for all specimens, including Specimen REF. Damage consisted of moderate spalling of the concrete cover and crushing of the core concrete over a plastic hinge region about 12 inches (305 mm) long. Spalling initiated at drift levels of 2.0, 2.0, 2.4, and 2.1 percent for specimens REF, LB-FB, LB-D1 and LB-D2, respectively.

At drift levels of 5.6, 5.3, 5.7, and 5.8 percent, respectively, the extreme flexural bars began to buckle, pushing out on the spiral. The spirals kinked and then fractured shortly after bar buckling was first observed. When the buckled bars were next loaded in tension, they straightened and fractured. The finding that buckling in the column occurred at almost the same drift in each specimen was surprising, given that in two specimens the bars were debonded over a length of 8 inches (203 mm) in the cap beam. However, the great majority of the bar deformation occurred in the column, where the detailing was the same in all specimens.

Bars in specimens LB-FB, LB-D1 and LB-D2 fractured at drift levels of 6.5, 7.1, and 7.4 percent, respectively. Bar fracture was brittle with no necking, and occurred approximately 6 inches (152 mm) above the interface with the cap beam, at the peak of the buckled shape. One bar fractured only partially during a load cycle and illustrated the failure process; the inside face of the buckled bar experiences extreme compression strain due to combined compression and bending. When the bar straightens, the inner face undergoes a large strain increment that initiates fracture. This observation and the lack of necking indicate that bar fracture occurs as a consequence of bar buckling and not tensile strain concentrations at the interface.

Despite their different capacities for inhibiting buckling, the two details, fully bonded and debonded, performed almost identically. This is confirmed by the similarity in their force-displacement curves in Figure 3. The primary reason was that the debonding was located in the cap beam, whereas the buckling deformations occurred in the body of the column, where both specimens were identical. It is felt that debonding is unnecessary, because the grouted bar pulls out a conical wedge of concrete from the duct, and, thus, this region behaves as an intentionally debonded bar permitting relief of strain concentration.

## Column-to-Footing Connection

**Concept:** The concept for the foundation is referred to as a socket column and is shown in Figure 4. The construction sequence is as follows. First the column is precast, with a roughened surface in the region that will eventually be embedded in the footing. Then the foundation is excavated and, in the bottom, a small slab is cast on which to set the column. The column is set, plumbed, leveled and braced, the footing steel is placed, and then the footing is cast.

The constructability advantages of the system are that it is quick and simple to build, the column is easy to transport because no hooked bars project from the bottom, it avoids any potential problems of fit-up of bars in ducts, the column detailing can be almost identical to that of a cast in place column, and no grouting is needed.

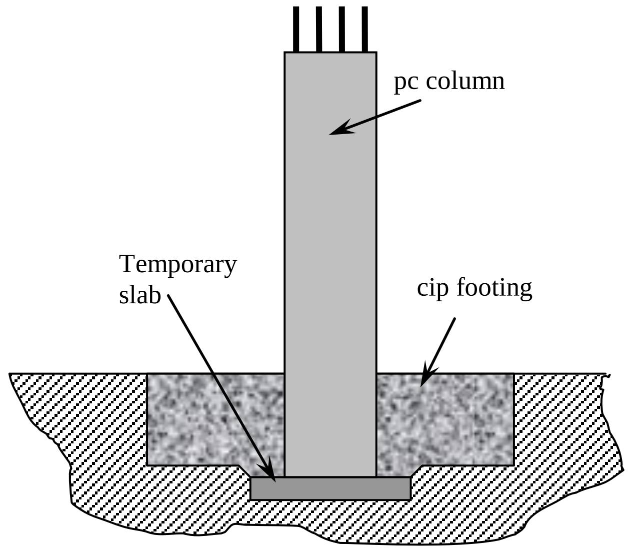



**Figure 4 – Socket Column: Concept**

The longitudinal reinforcement in the column is developed at the base by mechanical anchors, rather than the traditional method of bending the bars outwards. Doing so offers the construction advantage that the precast column becomes a large concrete cylinder with no protruding reinforcement, and is therefore simpler and safer to cast and transport. It also offers a much more direct transfer of forces between the column and the footing, as demonstrated by the strut-and tie models shown in Figure 5. Figure 5a (excerpted from Xiao et al., 1996) shows the model typically applied for bent-out bars. It requires extensive stirrup steel. Furthermore the hook on the main bar is ineffective for anchorage because it is facing the wrong way and, in fact, leaves an unreinforced section between the hooks and bottom bars. The model proposed for use with the headed bars (Figure 5b) is, by contrast, very simple and provides positive development of the reinforcement. The headed bars thus offer advantages for both constructability and seismic performance viewpoints. This is quite unusual: more commonly a change that benefits one imposes a disadvantage for the other.

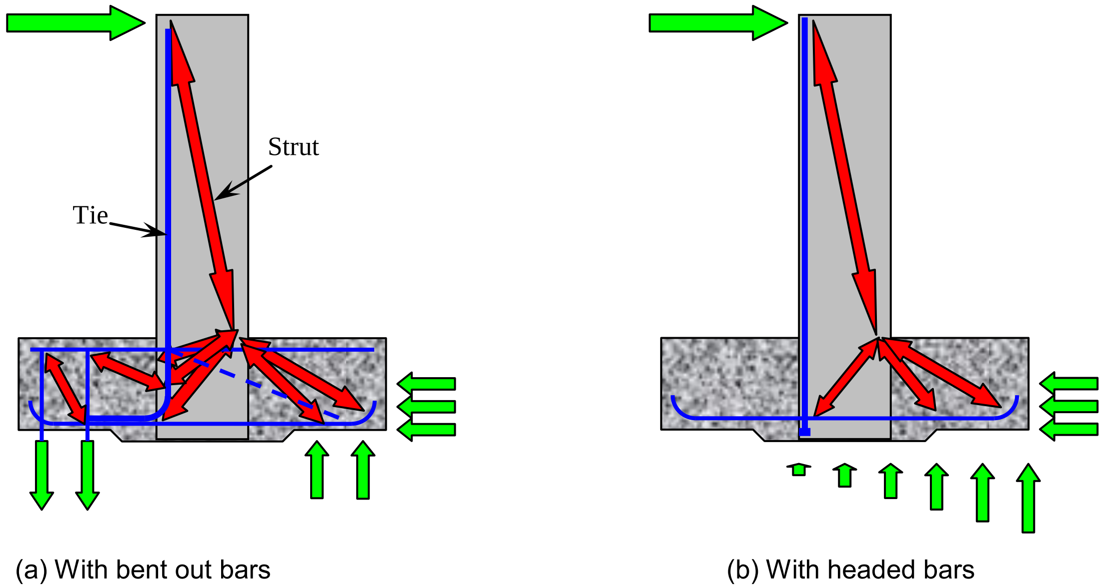
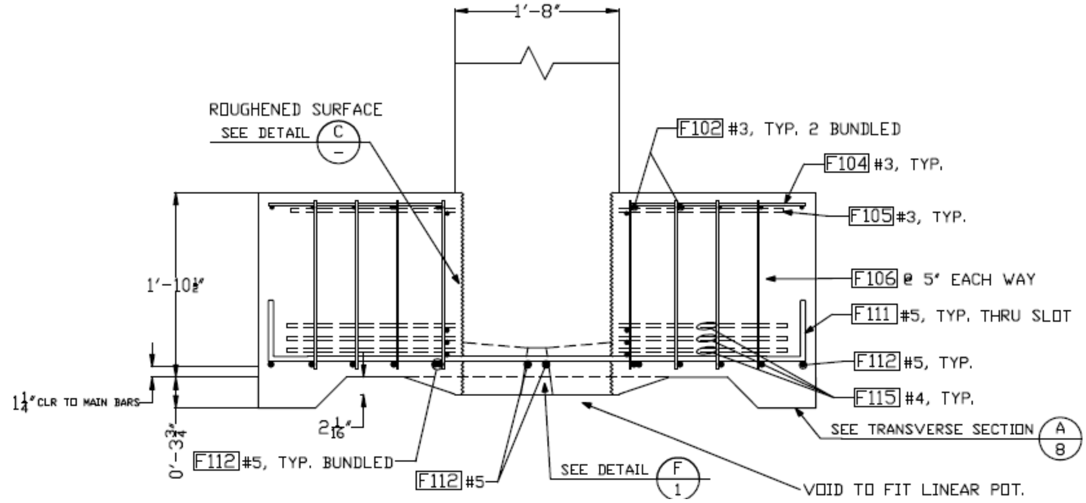


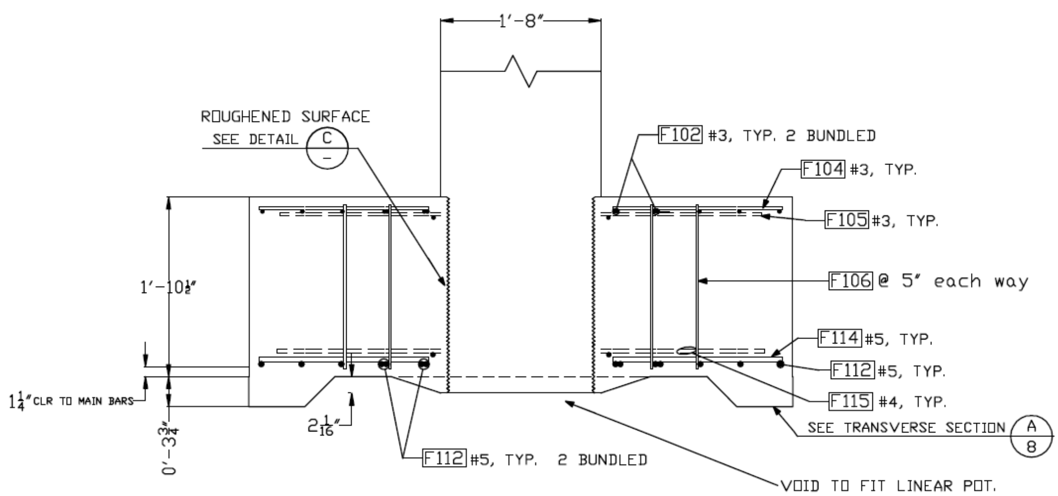

**Figure 5 - Strut and tie models for (a) bent out bars and (b) headed bars**

**Testing:** The major questions associated with the connection's seismic performance are expected to be in the transfer of forces from the column to the footing, so the testing focuses on them. The connection must be able to resist the cyclic column moments without significant damage to the footing, and without the column punching through the footing under gravity load. Osanai et al. (1996) tested socket columns for buildings and concluded that, unless special conditions were satisfied, the footing depth should be at least 1.5 times the column diameter. However, the sockets they used were much smaller in plan than a typical spread footing for a bridge, so the present tests, which are based on a footing depth/column diameter ratio of 1.0, are expected to demonstrate that the connection is strong enough to induce a plastic hinge into the column itself, just above the footing. (At the time of writing, the first test had just been completed. The specimen behaved exactly as expected). Achieving this behavior is important, not only because it is required by the AASHTO Specifications, but also because post-earthquake inspection and repair are more expensive for footings than for columns.

Details of the first two planned test specimens are shown in Figure 6. In both, the exterior of the column is roughened near the bottom to improve the transfer of shear stress. The column also extends just below the footing, to be sure that the force transfer at the node at the bottom of the column bars can take place satisfactorily.




**(a) Specimen A (Conservative)**




**(b) Specimen B (Simplified)**

**Figure 6 - Test specimens**

Specimen A is the more conservative option. Two footing bars run through a slot under the column in each direction to ensure their direct engagement with the tension steel in the column. Sets of diagonal bars in the horizontal plane are placed in the top and bottom of the footing around the column. Their purpose is to act as "shear friction" steel that will provide the friction forces needed to prevent punching failure along the precast/cast-in-place interface. One set is placed in the top and three stacked sets are placed in the bottom. Ties are included in accordance with the Caltrans recommendations (Caltrans, 2006). The "AASHTO Guide Specifications for LRFD Seismic Bridge Design" (AASHTO, 2009) are based on the Caltrans recommendations, but contain no tie requirements.

Specimen B was designed to determine whether the system could be simplified further. First, in each direction, the two center bars in the bottom mat of the footing steel were moved so they no longer pass under the column, but are placed just outside it. There they are bundled with the existing bars in that location. That placement frees the bottom mat to be placed at any time, thereby improving constructability. Second, the shear friction steel was reduced so that only one set of four bars was used at the top and bottom of the footing. The reasoning was that the bottom mat alone is sufficient, provided that it can be used to provide both flexural strength and shear friction. One set of diagonal bars was retained at the top and bottom to act as "trimming" reinforcement at the corners of the square opening in the footing mat. Last, the tie steel was reduced by 50 percent, on the basis that the Caltrans recommendations appeared to be developed for a system with main bars that are hooked outwards, rather than being anchored by heads. In that case the strut and tie model suggests that tie steel is largely unnecessary.

## Demonstration Project

The connections described above are designed to be used with a precast bent system that includes precast columns and a precast cap beam. Following the testing of the foundation connection and based on the success of the column-to-cap-beam connection, a demonstration project that uses these connections is planned by the Washington State Department of Transportation. The objective of the project is to demonstrate the constructability of the bent system on an actual bridge project that will be competitively bid and that crosses a major north-south freeway in Washington State, Interstate 5. The demonstration project is a replacement bridge that will be built on an alignment parallel to an existing bridge. It is two-span bridge with tall abutments on the end and a center bent that is located in the median strip. An elevation of the center bent is shown in Figure 7.

The superstructure of the bridge consists of 35-inch-deep (890 mm), decked-bulb tee prestressed girders that span 88 feet (26.9 meters). These are supported by the center bent composed of spread footings, precast column segments, a precast dropped cap beam and a cast-in-place diaphragm with a 5-inch (127 mm) cast-in-place topping over the decked bulb tees, whose flanges act as stay-in-place forms. The columns used in this project are spliced to permit erection in segments. While the columns of the demonstration project are small enough to be handled in a single pick with a crane, the segmental concept will demonstrate the technology for use on projects where the columns are longer and cannot be lifted with a single pick. Additionally, the laboratory specimens for the foundation connection include the same segmental connection, so that the capacity protection aspect of the system will be verified in the laboratory tests.

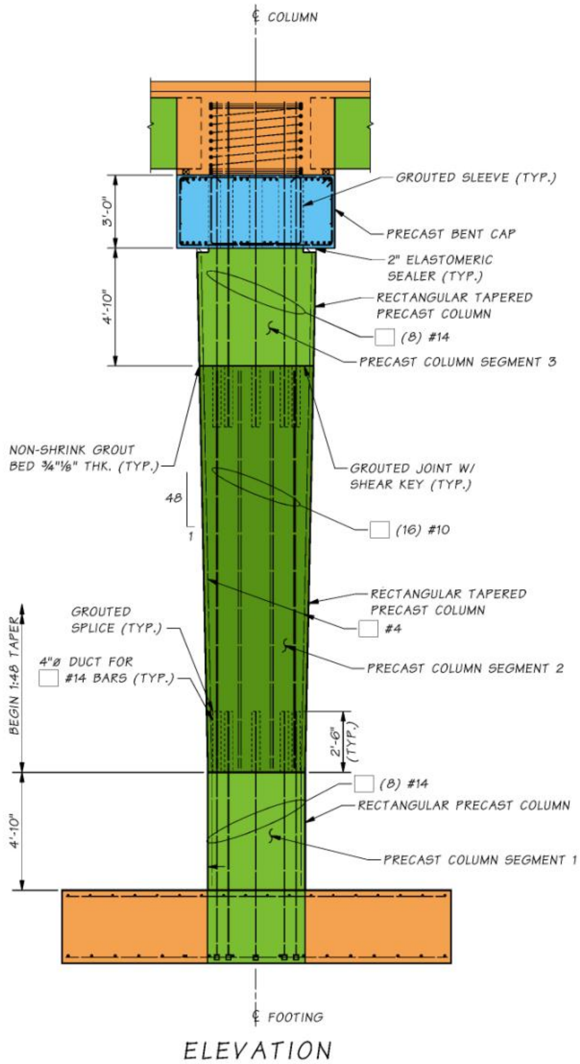



**Figure 7 - Demonstration Project Bent Configuration**

The precast bent system to be used in the demonstration project uses the common Washington State practice of integrating the prestressed girders with the integral full-depth diaphragm over a first-stage cap beam. This system provides longitudinal moment transfer from the bent columns to the superstructure. The precast first-stage cap beam for the demonstration bridge will be built in two pieces that are integrated with a closure pour near its mid-span. This is required because the bridge is 84 feet wide (25.6 meters), including sidewalks. Ideally, the precast first-stage cap would be built as a single piece element to avoid the time required for splicing segments, but lifting and shipping weight restrictions led to the two-piece solution. This decision could, of course, vary by project.

Along with the demonstration project, sections of the “AASHTO Guide Specifications for LRFD Seismic Bridge Design” that warrant changes in order to accommodate the proposed precast bent system will be identified and modifications will be proposed. Also, design examples for the use of precast bent systems with dropped cap beams that support prestressed girder superstructures will be developed. These design recommendations will encapsulate the findings of the study and allow the knowledge gained from it to be used in constructing future bridges more rapidly.

## Conclusions

A precast concrete bridge bent system is presented that is simple, rapid to construct and offers excellent seismic performance. The following conclusions are drawn.

1. The details have been developed with extensive input from a structural design engineer, the Washington State Department of Transportation, a precaster, and a general contractor. The assistance from a range of disciplines was critical to achieving the constructability goals.
2. The column-to-cap-beam connection is made with a small number of large (D57 or No. 18) bars column grouted into ducts in the cap beam. Their small number, and the correspondingly large ducts sizes that are possible, lead to a connection that can be assembled easily on site.
3. The column-to-cap-beam connection has been tested under cyclic loading in three variations. All three behaved essentially identically to a cast-in-place column with similar properties. All four specimens reached a drift of approximately 6 percent before bar buckling in the column precipitated failure.
4. The footing-to-column connection is being tested at the time of writing. One conservative option and one simplified option are being tested. Strut and tie analyses suggest that the conservative connection detail will be stronger than the column, and that failure will occur by plastic hinging in the column, as desired. The simplified detail uses less steel and is easier to construct. The constructability benefits are clear. Initial results from the tests show that the footing suffers no damage and that the seismic performance is as expected.
5. A demonstration project has been selected that will incorporate the precast bent system described herein to address constructability and illustrate feasibility for similar projects in the future. This demonstration, coupled with proposed design specification language and design examples, should facilitate the use of the precast bent system on similar precast girder bridges in seismically active regions.

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## References

"AASHTO Guide Specifications for LRFD Seismic Bridge Design" (2009). AASHTO, Washington DC.

Caltrans (2006). "Seismic Design Criteria Version 1.4." Caltrans, Sacramento, CA.

Getty Center Tram Guideway. http://www.cement.org/transit/tr_cs_gettycenter.asp

Matsumoto, E., Waggoner, M., Sumen, G., Kreger, M., Wood, S., and Breen, J. (2001). "Development of a Precast Bent Cap System," Center for Transportation Research, Research Project 0-1748, University of Texas at Austin.

Moustafa, S., (1989). "Ductile Pullout Connections," Concrete Technology Associates, CTA Bulletin 74B11. Now available from the Precast and Prestressed Concrete Institute, Chicago, IL.

Nakaki, S.D., Stanton, J.F., Sritharan, S. (1999). "Overview of the PRESSS Five Story Precast Test Building," PCI Journal, V.44 N2, March-April 1999, pp 26-39.

Osanai, Y., Watanabe, F. and Okamoto, S. (1996). "Stress Transfer Mechanism of Socket Base Connections with Precast Concrete Columns," ACI Structural Journal, ACI, Chicago. IL. May-June, pp. 266-276.

Pang, J.B.K., Eberhard, M.O., and Stanton, J.F. (2009). "Large-Bar Connection for Precast Bridge Bents in Seismic Regions," to appear in Journal of Bridge Engineering, ASCE.

Precast and Prestressed Concrete Institute (2004). "PCI Design Handbook," 6th ed. Chicago, IL.

Raynor, D.J., Lehman, D.L. and Stanton, J.F. (2002). "Bond-Slip Response of Reinforcing Bars Grouted in Ducts," ACI Str. Jo. 99(5), Sept. pp 568-576.

Steuck, K., Pang, J., Stanton, J. and Eberhard, M. (2007). “Anchorage of Large Bars in Grouted Ducts,” Washington State Transportation Center Report WA-RD 684.1, Seattle, WA.

Steuck, K., Stanton, J.F. and Eberhard, M.O. (2009). “Anchorage of Large-Diameter Reinforcing Bars in Ducts,” ACI Structural Journal, July-August, pp 506-513.

Xiao, Y., Priestley, M.J.N., and Seible, F. (1996). “Seismic Assessment and Retrofit of Bridge Column Footings,” ACI Structural Journal, ACI, Farmington Hills, MI. Jan-Feb., pp. 79-94.