



Bijan Khaleghi, Eric Schultz, Stephen Seguirant, Lee Marsh,  
Olafur Haraldsson, Marc Eberhard, and John Stanton

# Accelerated bridge construction in Washington State:

## From research to practice

- This paper describes the development and implementation of a precast concrete bridge bent system suitable for accelerated bridge construction in high seismic zones.
- At the base of the bent, the column is connected to a spread footing using a socket connection while at the top the column is joined to the cap beam using bars grouted in ducts.
- Both connections were tested at the University of Washington before the system was implemented on-site by the Washington State Department of Transportation.

**B**ridge construction frequently leads to traffic delays, which incur costs that can be measured in terms of time, wasted fuel, and emotional distress. Transportation agencies are therefore seeking methods for accelerated bridge construction (ABC). Use of precast concrete for bridge substructures offers potential time savings on-site and represents promising technology for ABC. Furthermore, limiting the amount of on-site work improves safety for both the motoring public and highway workers and reduces environmental impacts. For these reasons, transportation agencies are gradually embracing ABC for many of their urban construction projects.

Connections in precast concrete substructures 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 locations of high moments and shears and large inelastic cyclic strain reversals. Devising connections that can accommodate inelastic cyclic deformations and are readily constructible is the primary challenge for ABC in seismic regions. This paper describes the development, experimental validation, and implementation of a precast concrete bridge bent system that is intended to meet those challenges. This development was possible only by close cooperation among members of the team, which included the disciplines of design, research, precast concrete fabrication, and construction.

**Table 1.** Accelerated bridge construction decision-making matrix

No.	Question	Yes	Maybe	No
1	High traffic volume?			
2	Emergency replacement?			
3	Worker safety concerns?			
4	High daily traffic control costs?			
5	Evacuation route or over railroad or navigation channel?			
6	Lane closures or detours?			
7	Critical path of project?			
8	Closure possible during off-peak traffic?			
9	Rapid recovery/repair required?			
10	Adverse economic impact?			
11	Weather constraints?			
12	Environmentally sensitive site?			
13	Endangered species?			
14	Feasibility if historic bridge?			
15	Multiple similar spans (segments)?			
16	Problem for ready-mixed concrete?			
17	Delay-related user cost concern?			
18	Innovative contracting strategies?			
19	Adequate owner staffing?			

## Potential benefits of ABC and criteria for selection

The primary benefits of ABC accrue from saving time on site. Conventional bridge construction typically induces traffic congestion and extended delays. The traffic congestion adversely affects individual travelers' budgets and the region's economy, air quality due to increased vehicle emissions, and quality of life due to personal time delays. Also, untimely service due to delays for the workforce, suppliers, and customers can impose

significant costs on the traveling public and regional businesses.

Prefabrication of structural elements is the essence of accelerated construction. Although prefabrication can decrease total contract time, reduction of the time spent on-site is the critical component. Khaleghi<sup>1</sup> gives details of ABC use in Washington and the Washington State Department of Transportation's (WSDOT's) strategic plan for ABC.

Precast concrete units are often constructed in specialized plants. There, repetitive construction permits investment in high-quality steel forms, which facilitate high-quality finishes and accurate dimensional control. Plant precasting also allows tight quality control of materials, rapid production, good schedule control, and the possibility of prestressing. Site precasting offers other advantages, such as allowing workers to work at ground level and removing the need for, and limitations of transportation to the site. While precasting the substructure may impose a construction cost premium, it can often be offset by the economic benefits of the time saved through ABC.

As part of its ABC strategic plan, WSDOT has developed a preliminary decision-making matrix that is intended to identify at the project development stage the suitability of a bridge project for ABC (**Table 1**). Specific guidelines for the use of this matrix are still in development.

## Use of precast concrete substructures in seismic regions

For many years the State of Washington has designed and constructed precast, prestressed concrete girder superstructures because they have proved to be durable and cost effective. Girder technology has been continually improved so that spans in excess of 200 ft (61 m) are now possible.<sup>2</sup>

However, precast concrete substructures have seldom been used in high seismic regions, such as western Washington. Transverse seismic forces cause the largest moments to occur at connections (**Fig. 1**). Those connections must be moment resisting and robust under cyclic loading to maintain the integrity of the structure; if the members are precast, the connections must also be easy to assemble on-site. Achieving both characteristics simultaneously represents a significant design challenge.

In Washington, the cap beam is typically constructed in two stages. In a cast-in-place concrete bridge bent, the lower stage is cast on the columns, the girders are set on it, and finally the upper stage is cast with the deck slab.

Under longitudinal seismic loading, a moment connection between the girders and cap beam is desirable. Such a system is referred to as an integral bent cap and is commonly achieved by casting the upper-stage cap beam around bars and strands that project from the girder ends, thereby connecting them rigidly to the completed cap beam. In the absence of such a moment connection the columns must act as cantilevers, and such a system is not as efficient as one in which plastic hinging occurs at both the top and bottom of the columns.

## Design specifications and guidelines

There are two methods for seismic design of bridges: force-based design by the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*<sup>3</sup> and displacement-based design by the *AASHTO Guide Specification for LRFD Seismic Bridge Design*.<sup>4</sup>

WSDOT's seismic design is based on the AASHTO guide specification modified by the *WSDOT Bridge Design Manual*.<sup>5</sup> Displacement-based design is intended to achieve a no-collapse condition for bridges using one level of seismic safety evaluation. The fundamental design principle is capacity protection, where selected elements are identified for plastic hinging while others are protected against potential damage by providing them with sufficient strength to resist the forces consistent with the plastic hinge strengths.

Displacement-based analysis is an inelastic static analysis using the expected material properties of the modeled members. This methodology, commonly referred to as pushover analysis, is used to determine the reliable displacement capacity of a structure as it reaches its limit of structural stability.

The procedure outlined in the following steps is for displacement-based analysis and is applicable to bridges made of precast concrete components. The underlying assumption is that the displacement demand obtained from linear-elastic response spectrum analysis can be used to estimate the displacement demand even if there is considerable nonlinear plastic hinging.

1. Develop an analytical model with appropriate foundation stiffness and yielding member stiffness based on moment-curvature relationships. For capacity-protected members, including the precast concrete girder-to-diaphragm connection, consider the properties of the cracked section.

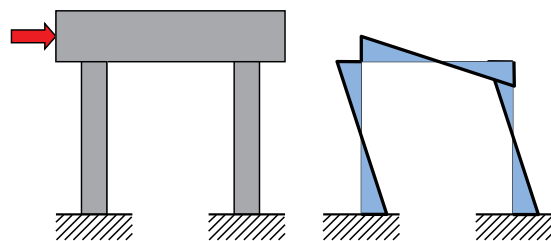


Figure 1. Moment diagram of a bridge pier with fixed connections.

2. Perform linear elastic response spectrum analysis of the bridge based on design acceleration spectra given in national or local specifications.
3. Determine the lateral and longitudinal displacement demands at each pier, including appropriate directional combinations.
4. Perform pushover analysis of each pier in the local transverse longitudinal directions. For this purpose, the plastic hinging behavior for each column must be included, and this will generally be based on the moment-curvature relationships used in step 1. Use foundation stiffnesses that are consistent with those used in the displacement demand model.
5. Compare the total displacement capacity of the pier, based on concrete and steel strain limits, with the displacement demand. Also compare the displacement ductility demand with the permissible capacity. If either the displacement or ductility capacity is insufficient, revise accordingly.
6. Capacity protect the superstructure and foundation for the overstrength forces (typically, 20% higher than the plastic capacity of the columns) to make sure that plastic hinges occur within the column. Capacity protect the column in shear for these same overstrength forces.

## Bridge bent configuration selected

Figure 2 shows the configuration of the bridge bent system that was developed. It consists of a cast-in-place concrete spread footing, a precast concrete column, and a precast concrete first-stage cap beam. The second-stage cap beam is cast in place, just as it would be in a fully cast-in-place concrete system. The footing-to-column and column-to-cap beam connections are the critical elements that lead to the system's viability, and the genesis of each is reviewed here.

The footing-to-column connection is referred to as a socket connection (Fig. 3). It is made by placing the precast concrete column in the excavation, placing the footing steel, then casting the footing concrete. Alternatively,

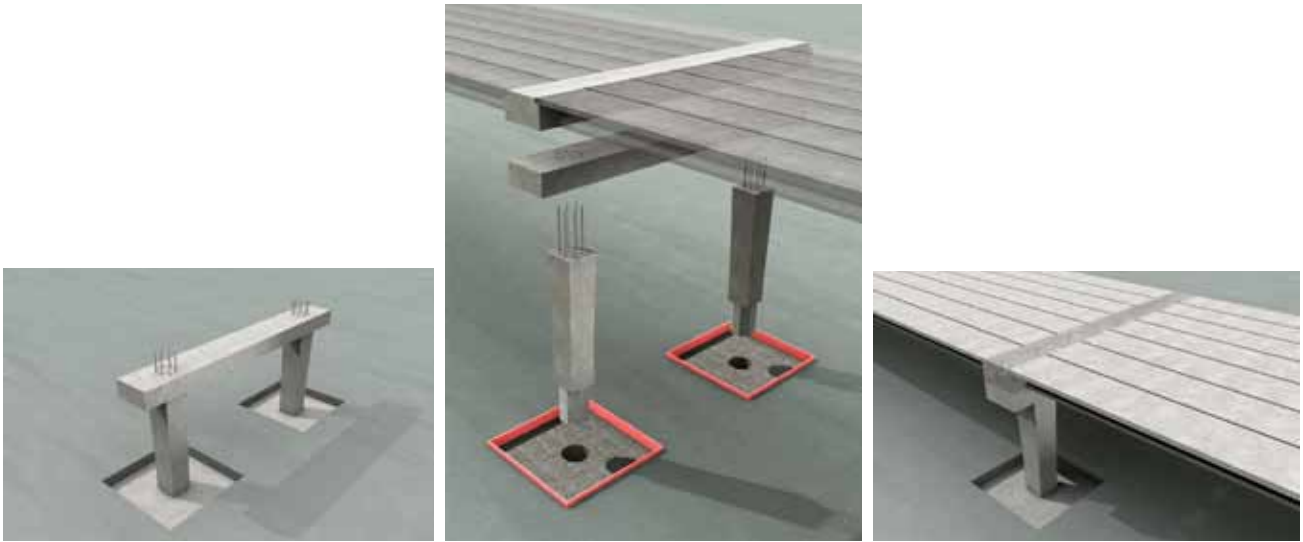


Figure 2. Precast concrete bent system configuration.



Figure 3. Socket connection concept.

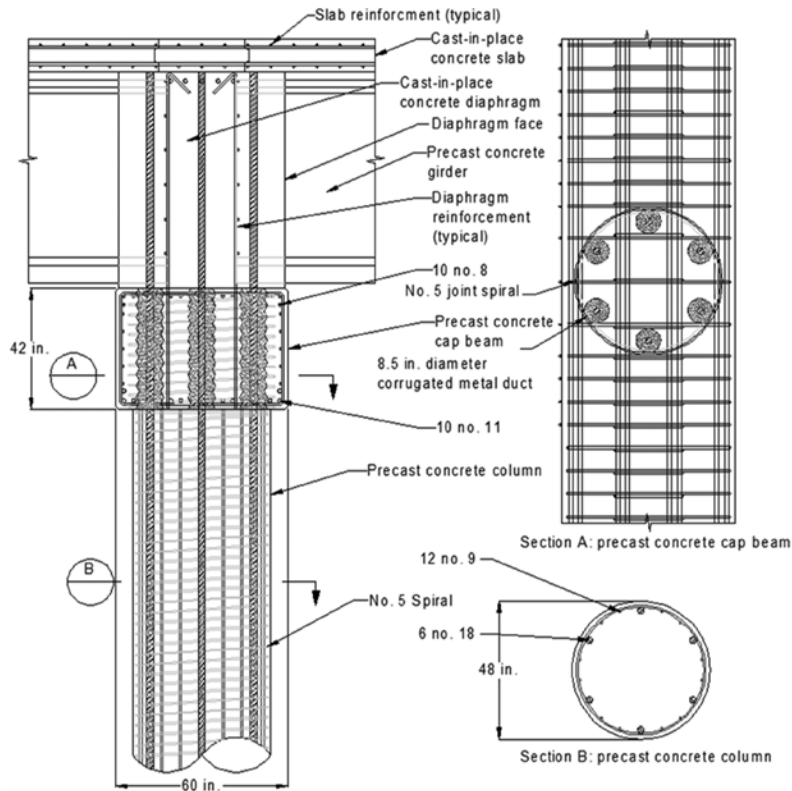


Figure 4. Previous use of precast concrete cap beam that used a large number of column bars in a seismic area of Washington State.

the footing steel may be placed before the column is set. The precast concrete column-to-footing connection's primary advantage is construction speed because it allows a footing and a column to be cast in little more time than that needed to cast a footing alone. Furthermore, because the finished connection can carry vertical forces greater than the weight of the cap beam, the footing needs to gain only a fraction of its full strength before the cap beam can be placed. The time to the start of setting girders on the cap beam is a critical measure of the savings provided by the bent system.

The socket concept was used previously in Washington in a modified form. In that case, the contract called for cast-in-place concrete columns, but the contractor elected to precast them on-site and use a socket connection to save time. The footing was 6 ft (1.8 m) thick, the columns were 4 ft (1.2 m) square, and the connection between them was made by roughening the column surface locally and adding horizontal form-saver bars. Those bars screwed into threaded couplers embedded in the face of the column within the depth of the footing to provide shear friction across the interface and were inserted after the column had been placed.

The column-to-cap beam connection was made with vertical bars projecting from the column that were grouted into ducts in the cap beam. Again, this concept has been used previously, but primarily in regions of low seismicity where the number of bars needed for the connection was small and the loading was not cyclic. The concept was also used once in the high seismic zone in western Washington. Figure 4 shows fabrication and subsequent placement of that precast concrete cap beam. The bridge site is in a con-



**Figure 5.** Column-to-cap beam connection. Note: no. 5 = 16M; no. 8 = 25M; no. 9 = 29M; no. 11 = 36M; no. 18 = 57M; 1 in. = 25.4 mm.

gested urban area with high visibility from the traveling public and high scrutiny from associated municipalities. To open the bridge as quickly as possible, the contractor proposed precasting the cap beams for the intermediate piers instead of casting them in place as shown on the contract plans. This change saved the owner and the contractor several weeks. The columns were reinforced with the same fourteen no. 14 (43M) column bars as on the original plans. They were grouted into 4 in. (100 mm) galvanized steel ducts that were placed in the precast concrete cap beam using a template. The cap beams weighed approximately 200 kip (890 kN) each and were precast on the ground adjacent to the columns.

For the precast concrete bent system described in this paper, the grouted bar-beam connection was modified by using the largest bars possible, up to and including no. 18 (57M) bars. That choice allows the ducts to be large in diameter and few in number; both features facilitate fit-up on-site and reduce the probability of accidental misalignment. However, anchorage of such large bars within the depth of the cap beam is not possible if the development length equations of the AASHTO LRFD specifications must be satisfied. Previous studies<sup>6</sup> had indicated that bars grouted into ducts resulted in significantly shorter development lengths than predicted by the standard

equations due to the confinement provided by the duct, but those studies examined smaller bars and tighter ducts than proposed here. Research was therefore undertaken to determine the development properties of large bars grouted into large-diameter ducts<sup>7</sup> and the response of such connections to cyclic lateral loading.<sup>8</sup> That research is described in detail in the following paragraphs.

**Figure 5** shows the cap beam-to-column connection for the proposed system. The precast concrete column has six no. 18 (57M) vertical column bars that project from its top. The precast concrete cap beam, which contains 8 in. (200 mm) diameter corrugated metal ducts, is fitted over the column bars and grouted in place, completing the bent. The selection of six no. 18 vertical column bars reduces the congestion at the column-to-cap beam connection while providing generous assembly tolerances.

The top and bottom connections are different because although the seismic performance requirements are similar in both locations, the construction needs are not. A spread footing for a typical overpass is generally too heavy for precasting to be viable, so it is likely to be cast in place. Then, the socket connection provides generous tolerances and fast construction. However, a socket connection at the top would require casting the cap beam in place, and that would eliminate much of the time advantage of prefabrication. Thus a socket connection at

the base and a grouted-duct connection at the top were selected as practical solutions to this problem.

The connections may be compared with other alternatives, such as those given in Marsh et al.<sup>9</sup> For example, grouted sleeves have been adopted for the base connection by a number of agencies, such as the Utah Department of Transportation. The sleeves are typically cast into the column and fit over bars projecting upward from the footing. The socket system proposed here has the advantages that the placement tolerances for the column are significantly greater than those available with a commercially available sleeve system, and the connection requires no special or proprietary hardware.

## Supporting research: Cap beam connection

The major questions about the system that required investigation concerned the connections. At the cap beam, the dominant concerns were the anchorage of large bars in ducts and the inelastic cyclic performance of a moment connection made with large bars.

The bar anchorage demands can be divided into two categories. For the first-stage precast concrete cap beam, the length available for bar development is limited by the depth of the cap beam, and the loads consist of the weight of the girders and slab. (The second-stage cap beam is typically cast with the last section of slab, so most of the slab weight will be in place before the second stage is cast). Because all the girders on one side of the cap beam may be placed before any are set on the other side, the cap beam may experience torsion. This torsion may result in tension in some of the bars, which controls the development demand in the first-stage cap beam. In the great majority of cases, anchorage sufficient to develop the yield strength of the bar would be sufficient to resist the construction loads.

Seismic loading leads to higher bar stresses and requires better anchorage, but for all but the largest and most critical bridges, seismic design is conducted only for the completed bridge, in which case both stages of the cap beam are in place and the total height of the cap beam is available for development of the column bars. Thus, at that time, both the bond demand and the total bond capacity are larger. However, the components of the total bond capacity in the two stages of the cap beam are likely to differ because the grouted ducts exist only in the first-stage cap beam.

To investigate the development of bars grouted in corrugated steel ducts, 14 monotonic pullout tests were performed with bars as large as no. 18 (57M).<sup>7</sup> They supplemented a previous test series at a smaller scale.<sup>6</sup>

The material characteristics in the tests included ASTM A706<sup>10</sup> Grade 60 (410 MPa) deformed reinforcing bars, corrugated galvanized pipes, and cementitious grout with compressive strength of 8.0 ksi (56 MPa). The

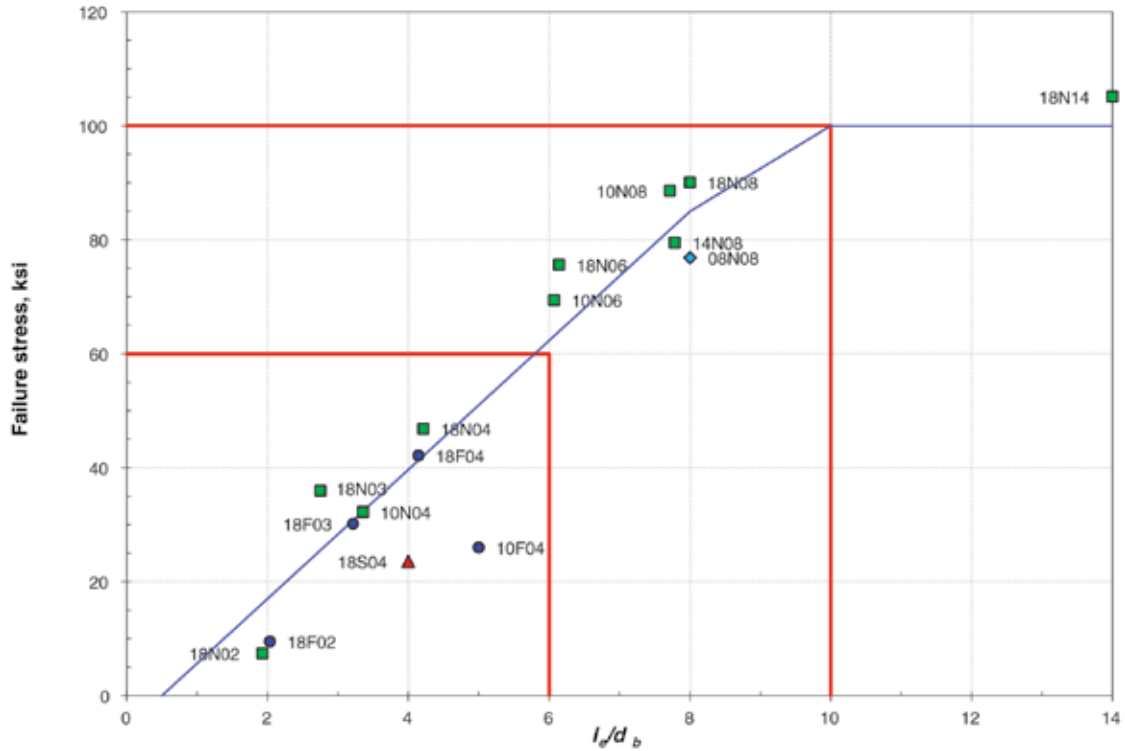
corrugated pipes are available in diameters from 6 in. (150 mm) to 12 ft (3.7 m). The pipes have thicker walls, deeper corrugations, and potentially better bond and confinement properties than those of standard posttensioning ducts.

**Figure 6** summarizes the results of the pullout tests. It shows the bar stress at failure plotted against the ratio of embedment length to bar diameter  $l_e/d_b$  to permit comparison among different bar sizes. In the nomenclature for the tests, 18N06 means a no. 18 (57M) bar with no fiber in the grout embedded 6 bar diameters. The letter F signifies fibers in the grout, N signifies no fibers, and S indicates a failure near the surface, which was controlled by a tension failure cone in the concrete surrounding the duct, rather than a shear failure in the grout.

The fibers were polypropylene with a dosage of 3 lb/yd<sup>3</sup> (1.8 kg/m<sup>3</sup>). They were used in some pullout specimens, but they adversely affected the grout strength and therefore the anchorage performance, so they were not used in the final connection. A non-linear numerical model was calibrated against the test results, and the model's results are also shown. Finally, separate lines show the nominal yield and ultimate stresses of the bars.

Three outcomes can be seen from the tests. First, the bar stress at failure is essentially proportional to  $l_e/d_b$ . This implies that the bond stress is constant along the bar and the same in all specimens and that failure was by plastic shear failure in the grout. Visual observations supported that finding. Second, the bar can be anchored to reach yield and fracture if the embedment lengths are  $6d_b$  and  $10d_b$ , respectively. These are experimental results, and suitable safety factors should be added for design, particularly for cyclic loading. However, the results show that even a no. 18 (57M) bar can easily be anchored to achieve fracture within the 3.5 ft (1.1 m) depth of a typical cap beam. Third, the presence of fibers in the grout actually reduced the bond strength as well as the cube strength of the grout. Inelastic elongation of the bar is accompanied by a reduction in its diameter, which causes the lugs to partially disengage from the surrounding grout, thereby reducing the bond capacity. In the post-peak region, the fibers had been expected to improve the behavior by bridging cracks in the grout, but the ducts appeared to fulfill that function. When the bond strengths were normalized by the square root of the grout compressive strength, the peak resistance of the specimens with and without fibers was similar. It was concluded that the duct provided adequate confinement to the grout and that the fibers were superfluous.

Once the anchorage properties under monotonic tension loading had been established, column-to-cap beam connection tests were conducted under cyclic lateral loading.<sup>8</sup> **Figure 7** shows a typical test. The



**Figure 6.** Grouted bar-duct pullout test results. Note:  $d_b$  = bar diameter;  $l_e$  = embedment length. 1 ksi = 6.895 MPa.

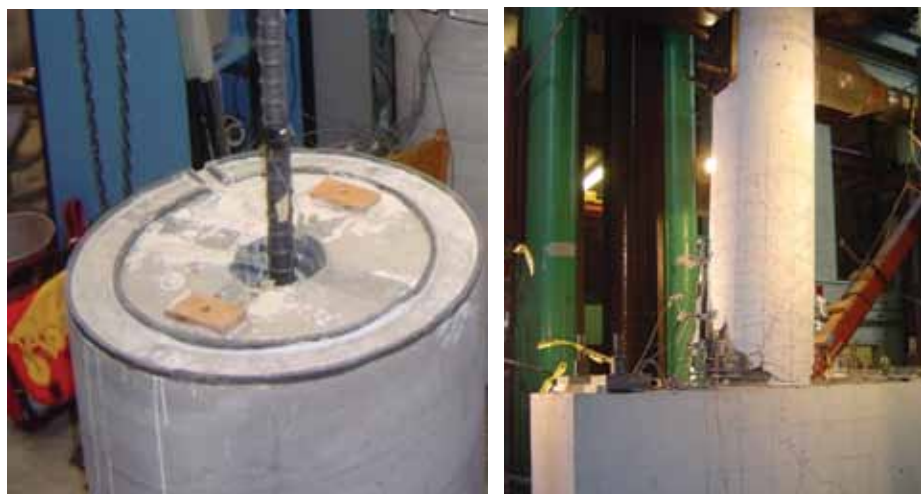
specimens were tested upside down so that the cap beam could be bolted to the base of the test rig. The specimens were 42% scale, so the 20 in. (500 mm) test column represented a 48 in. (1200 mm) prototype. The goal was to investigate the behavior of complete grouted bar connections under cyclic lateral load.

The cyclic tests were performed on three variations of the large bar precast concrete system, as well as a typical cast-in-place concrete connection for comparison. All three variations of the proposed system performed satisfactorily to a drift ratio of 5.5%, after which longitudinal bar buckling and fracture occurred. This value is approximately three times the demand expected in a major earthquake and is comparable to the value achieved with a cast-in-place concrete system. In all cases the failure occurred in the plastic hinge region of the column. This finding suggests that the large-bar, large-duct precast concrete system has sufficient strength and ductil-

ity capacity for all foreseeable seismic demands and system performance is similar to that of cast-in-place concrete construction.

## Supporting research: Spread footing connection

Additional research by the team had a goal of combining the upper connection and the socket footing connection into a complete bent, which would be taken to the point of implementation. To achieve that goal, three socket connections were tested in the laboratory,<sup>11</sup> and a demonstration bridge was then constructed with the bridge bent system over Interstate 5 (I-5).<sup>12,13</sup>



**Figure 7.** Test setups of large bar-duct pullout and column-to-cap beam connection.



**Figure 8.** Construction and testing of precast concrete column-to-footing connection.

The goal of the laboratory tests was to evaluate the connection's response to combined cyclic lateral load and constant vertical load. The test specimens consisted of 20 in. (500 mm) diameter precast concrete columns embedded in cast-in-place concrete foundations. The columns were cantilevers and were loaded at a location that corresponded to the inflection point in the prototype column. The cantilever height was 60 in. (1500 mm), or three column diameters. **Figure 8** illustrates the construction and testing.

In each of the first two tests, the column contained a splice. The purpose was to determine where splices could be located in the event that constructability constraints in some future project might require a segmental column. The splice detail was an optional feature of the bent to permit the use of taller columns.

In all three cases, the column surface was roughened where it was embedded in the footing. The roughening was achieved using small timber strips that represented, at laboratory scale, the sawtooth pattern used on the ends of standard Washington prestressed concrete girders. Use of timber strips necessitated flat surfaces, so the cross section was changed from circular to octagonal in that portion of the column. Other methods of surface roughening, such as sandblasting or using a surface retarder,

may be possible and would allow the use of a circular section throughout, but the resulting degree of roughness would need to be verified.

In the first two spread footing specimens, SF-1 and SF-2, the footing depth was approximately equal to the column diameter. These proportions are typical of cast-in-place concrete construction. Those two specimens failed in the column with no damage at all to the footing, so a third specimen was constructed with a footing depth that was only half the column diameter. The goal was to force failure into the connection region to gain a better understanding of the flow of forces there and the possible failure mechanisms.

The tests are described in detail in Haraldsson et al.<sup>12</sup> The results showed the following:

- Under cyclic lateral loading, the precast concrete column and socket connection perform as well as, or better than, a comparable cast-in-place concrete system if the footing depth is at least equal to the column diameter.
- The connection can resist, without damage, the maximum probable vertical load.
- No reinforcement is needed across the column-footing interface. The column bars are terminated with a mechanical anchor rather



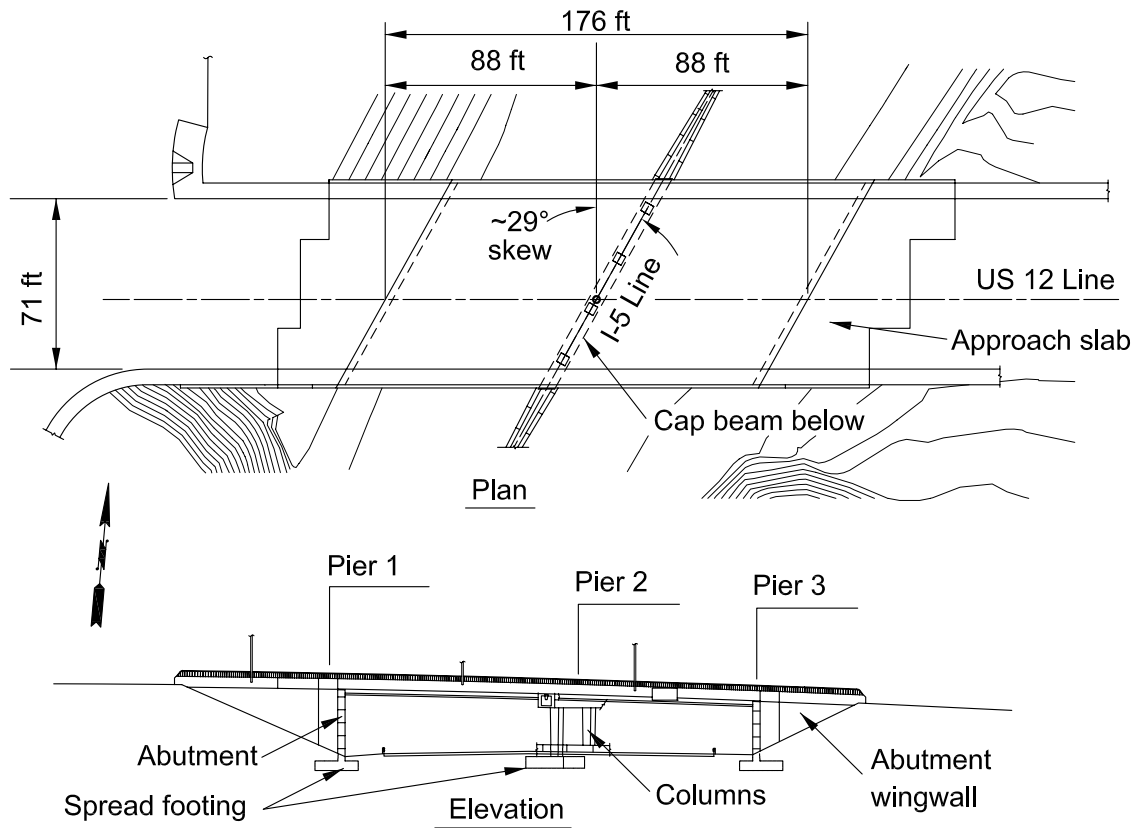


Figure 9. Bridge layout for demonstration project. Note: Elevations are in feet. 1 ft = 0.305 m.

than a hook, and the footing reinforcement that normally passes beneath the column may be moved laterally and bundled with the other bars adjacent to the column. The same may be done with the top footing reinforcement. Mechanical anchors should comply with the requirements of *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)* appendix D.<sup>14</sup>

- The footing ties required by the AASHTO guide specification<sup>4</sup> are not needed if the column bars are equipped with anchor heads that are set just below the bottom footing reinforcement. The anchor heads permit the formation of a simple strut-and-tie system within the footing and inhibit diagonal cracking and joint shear failure. It is in this respect that the behavior is potentially better than that of a typical cast-in-place concrete connection system constructed with bent-out bars, for which the flow of forces is more complicated and footing ties are needed.

## Site implementation

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 was planned and executed.<sup>12</sup> The objective of the project was to demonstrate the constructability of the bent system on a bridge project that would be competitively bid. The demonstration project is a replacement bridge that was built on an alignment parallel to an existing bridge and crosses I-5 in Washington. The bridge has two spans, tall abutments at each end, and a center bent that is located in the median strip of the freeway. **Figures 9** through **12** show the details of this project.

The bridge features include the following:

- unique socket connection of precast concrete column to footing
- precast concrete columns fabricated in segments and joined by bars grouted in ducts
- precast concrete cap beam made in two segments that were joined by a cast-in-place concrete closure
- precast concrete superstructure with cast-in-place concrete closure at intermediate pier
- precast concrete end and intermediate diaphragms

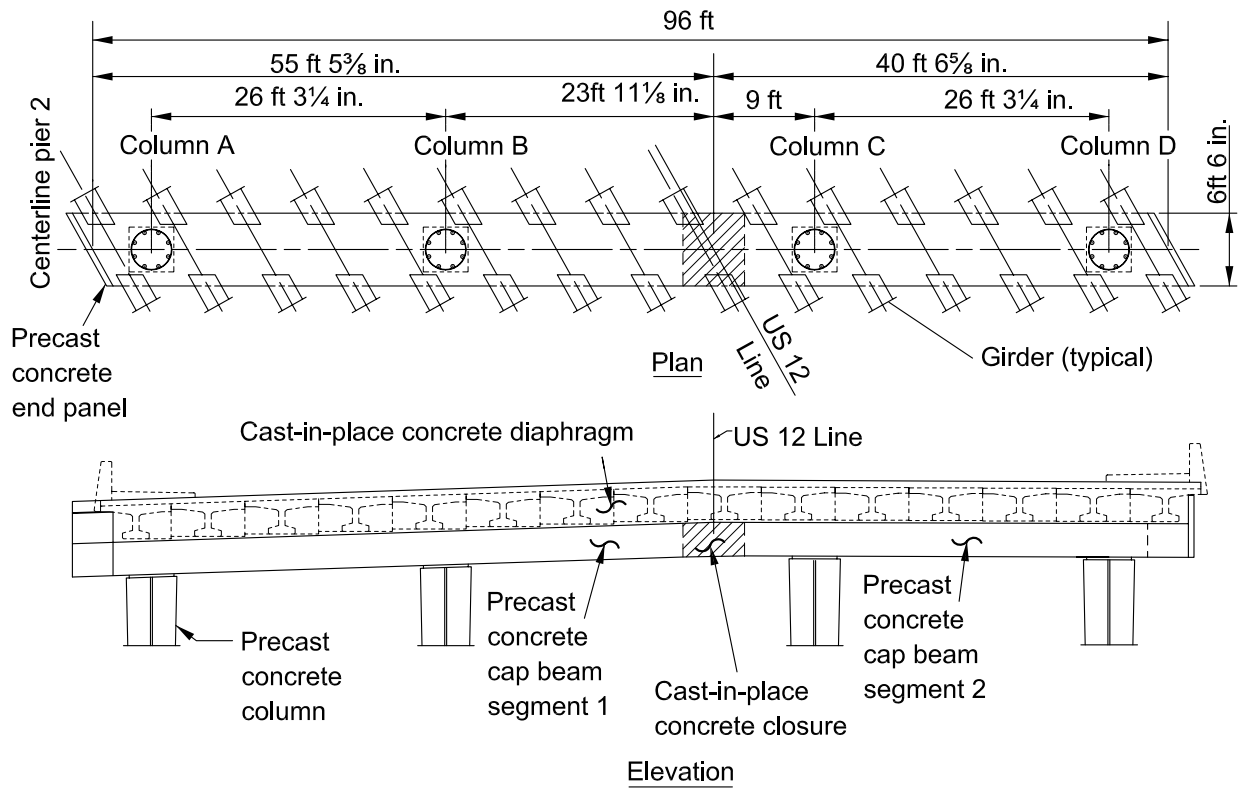


Figure 10. Bridge plan and elevation at pier for demonstration project. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

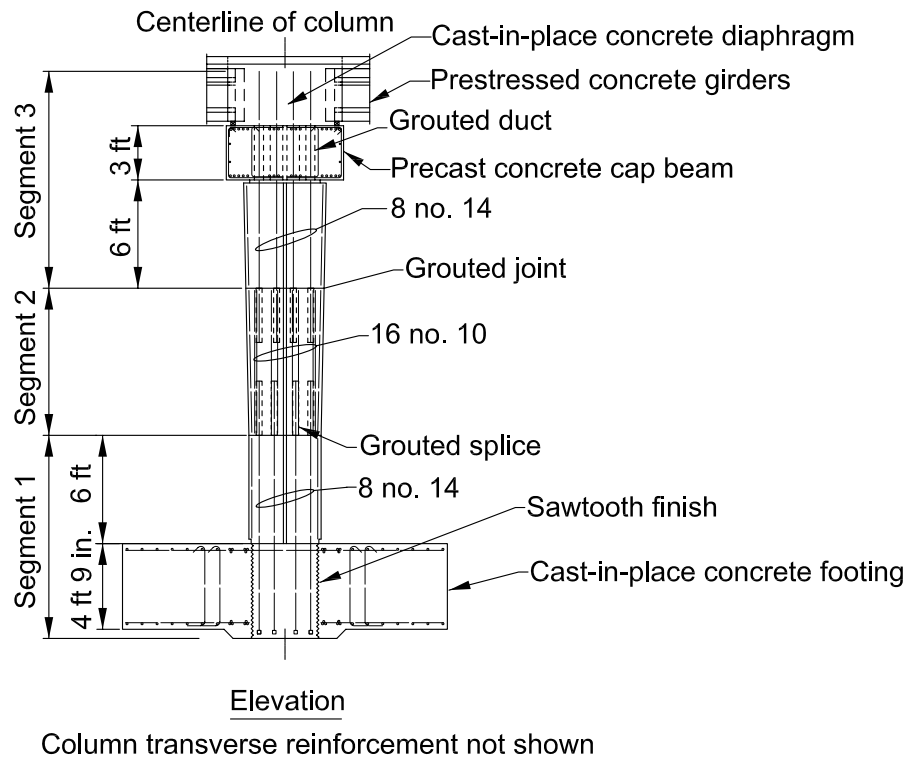
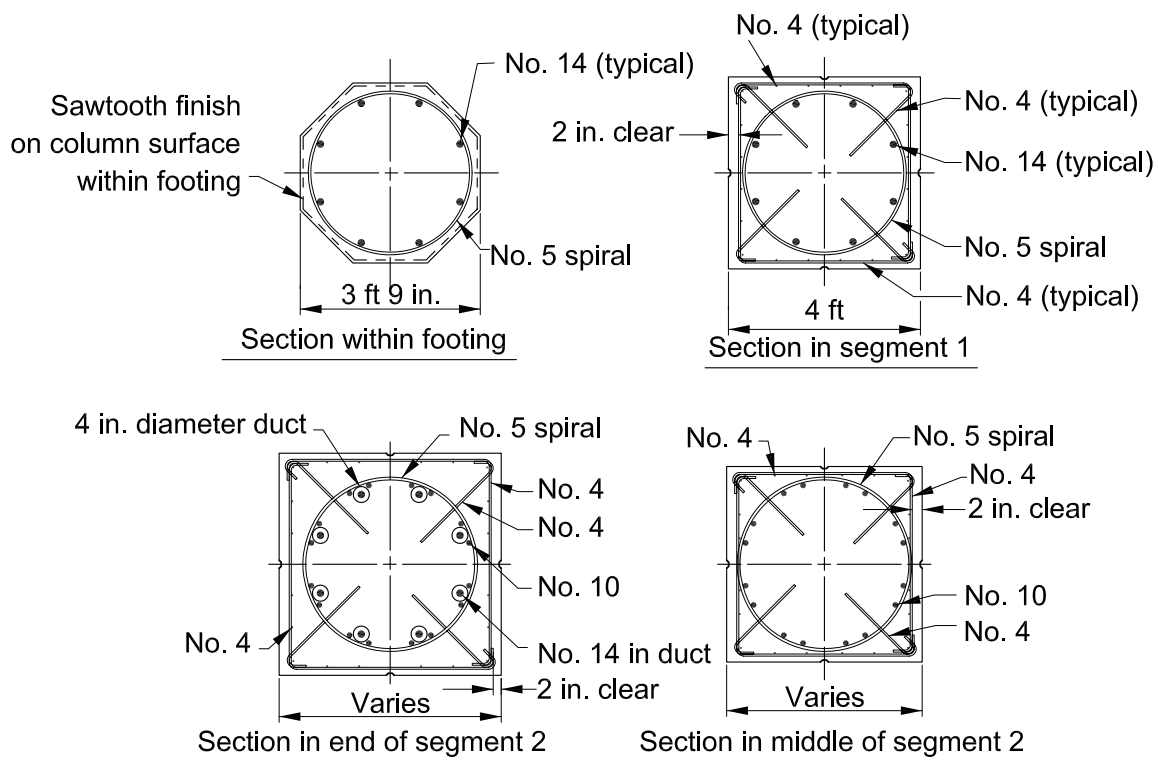


Figure 11. Demonstration bridge column details for elevation. Note: no. 10 = 32M; no. 14 = 43M; 1 in. = 25.4 mm; 1 ft = 0.305 m.



**Figure 12.** Demonstration bridge segmental column detail sections. Note: no. 4 = 13M; no. 5 = 16M; no. 10 = 32M; no. 14 = 43M; 1 in. = 25.4 mm; 1 ft = 0.305 m.

The details of the new connections were essentially identical, apart from scale, to those tested in the laboratory.

The steps in the construction sequence for the column-to-footing connection are listed (**Fig. 13**):

1. Excavate for footing and install forms.
2. Place leveling pad, and set first segment of column.
3. Place footing reinforcing, and cast footing concrete.
4. Remove forms, and backfill around the excavation.

The columns used in this project were fabricated in segments and spliced on-site. Although the columns of the demonstration project were small enough to be handled as a single piece, the segmental concept was used to demonstrate the technology for use on other projects where the columns are larger and cannot be transported or lifted as a single piece.

Following are steps in the construction sequence for placement of precast concrete column segments and cap beam (**Fig. 14** and **15**):

1. Place and shim middle column segments.
2. Place and shim top column segments.
3. Install column bracing.
4. Place and shim precast concrete cap beam segments.
5. Grout all ducts and segment interfaces.

The precast concrete bent system used in the demonstration project relied on the standard Washington practice of integrating the prestressed concrete girders with the cast-in-place concrete second-stage cap beam.



**Figure 13.** Construction sequences for placement of precast concrete column segment into footing.



**Figure 14.** Placement of column segment.

This system provides longitudinal moment transfer from the bent columns through the cap beam to the girders. The precast concrete first-stage cap beam for the demonstration bridge was built in two pieces that were integrated with a closure near midwidth of the bridge. This was required because the bridge is 84 ft (25.6 m) wide, including sidewalks. Ideally, the precast concrete first-stage cap would be built as a single piece to avoid the time required for splicing segments, but lifting and shipping weight restrictions led to the two-piece solution in this case. This decision will vary by project.

The joints between column segments and the column-to-cap beam were all grouted at one time. The grouting process included the following steps:

1. Install grout forms and seal.
2. Pump grout and close grout tubes.
3. Remove grout forms and inspect grout in joint and grout tubes.

4. Repair unfilled grout tubes and patch back grout tubes.

Because of the relatively small size of the column and precast concrete cap beam segments, all pieces on this bent could be stacked and braced before any joint grouting was necessary, and this approach minimized the number of separate grouting operations. This would not necessarily be the case if larger segments were required, such as might be expected in taller column segments. In those cases, intermediate grouting steps would be necessary to ensure structural stability during construction.

They were grouted with the joint interfaces in one operation per interface. In each case a dam was formed around the joint and grout was pumped in at the bottom until it emerged from the upper vent port(s). Because in the column segments all the ducts were located in the middle segment, the inlet port in the lower interface was in the dam, and the vents were

at the tops of the ducts, while in the upper interface those locations were reversed.

The superstructure of the bridge comprises 35 in. (890 mm) deep decked bulb-tee prestressed concrete girders that span 88 ft (26.8 m). These are supported by the center bent, which consists of spread footings, precast concrete column segments, a precast concrete dropped cap beam, and a cast-in-place concrete diaphragm. A 5 in. (125 mm) cast-in-place concrete topping is placed over the decked bulb tees, whose flanges act as stay-in-place forms.

The construction sequence for placement of the precast concrete superstructure at the intermediate pier (Fig. 16) is as follows:

- Place precast concrete girders on oak blocks.
- Install girder bracing as necessary.
- Complete welded ties between girders.
- Join flange shear keys and grout intermediate diaphragms.
- Place slab reinforcement and cast concrete.
- Cast pier diaphragm concrete 10 days after slab casting. Each deck bulb tee was fitted with precast concrete transverse end walls to serve as side forms for the cast-in-place concrete pier diaphragm.
- Cast traffic barrier and sidewalk.

## Lessons learned

The demonstration project was built successfully without major setbacks or unexpected events. The times for individual operations were recorded, but no effort was made to compare them with those for traditional methods because this was the first time the system had been built and the primary interest lay in resolving any construction difficulties that might occur. The primary challenges that were encountered were associated with grouting the column segments



Figure 15. Placement of precast concrete cap beam.

and making the closure in the cap beam. Because the segmental columns were not an essential part of the new system, and the contractor's comments would apply to any grouted joint, they are not addressed in detail here. However, further experience with constructing this type of joint, which is common in precast concrete construction, would alleviate the difficulties experienced on this project.

The following feedback was provided by the bridge contractor and others involved in the construction of the demonstration project:

- The use of a single-piece precast concrete column instead of cast-in-place concrete columns offers the potential for time savings.
- Grouting all the joints at once maximized the weight acting across the interfaces, which in turn allowed the use of a high-pressure grout pump without risk of accidentally lifting some seg-



Figure 16. Placement of precast concrete girders and casting slab concrete.



ments. A high-pressure pump was seen as desirable to ensure that all ducts were filled. However, the ducts in the spliced column tested in the laboratory were filled without difficulty using a low-pressure system.

- The grouted joint between the column and the cap beam was the easiest to construct. It would have been even easier had the joint been wider.
- The reinforcement in the cap beam closure region was congested and, had it not been for close cooperation with, and careful detailing by, the precaster, bar interference might have occurred. In practice, all longitudinal bars fitted without any interference.
- The stirrups in the cap beam closure region were detailed as U bars, but placement would have been easier if they had been detailed as a series of J hooks.
- The closure region was congested, and concrete consolidation would have been easier with a pea gravel concrete mixture. The hanging formwork for the closure was simple and easy to construct.
- The 35 in. (890 mm) deck bulb tees were fabricated with precast concrete transverse wall segments at the central-pier end of the girders with the goal of saving the time needed to construct on-site formwork for the second-stage cap beam. However, the contractor found that those 5 in. (130 mm) precast concrete wall elements were not useful because of the complexity of skew, cross slope, and tolerances. Formwork still had to be installed between the bottom of these walls and the top of the precast concrete cap beam and around the oak blocks, and it would have been simpler to complete the entire second-stage cap using timber formwork without the precast concrete walls on the girder ends.

## Additional supporting information

In addition to the spread footing concept used in the demonstration project bridge over I-5, a precast concrete column-to-drilled shaft connection has been developed and is now being tested. Adaptation of the precast concrete bent system to this widely used foundation type should extend its potential range of use. Also, design examples and example specifications are being developed as part of the demonstration project.<sup>15</sup>

## Conclusion

A precast concrete bridge bent system is presented that is conceptually simple, can be constructed rapidly, and offers excellent seismic performance. The following conclusions are drawn:

- The system described here addresses the demands of both seismic performance and constructability. It provides an example of a successful transfer of research to practice but was possible only through the close cooperation between team members representing research, design, fabrication, and construction.
- Precast concrete bridge systems are an economical and effective means for rapid bridge construction. Precasting eliminates traffic disruptions during bridge construction while maintaining quality and long-term performance.
- The use of precast concrete cap beams results in time and cost savings by eliminating the need for elevated falsework and shoring. It also improves worker safety because reinforcement and concrete can be placed at ground level.
- The column-to-cap beam connection is made with a small number of large bars 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.
- The development length of a reinforcing bar grouted into a corrugated steel pipe is much shorter than implied by current code equations for a bar embedded directly in concrete.
- The socket connection between the cast-in-place spread footing and the precast concrete column provides excellent performance under combined constant vertical and cyclic lateral loading and is quick and easy to construct.

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# Notation

- $d_b$  = bar diameter
- $f_u$  = specified tensile strength of reinforcement
- $f_y$  = specified yield strength of reinforcement
- $l_e$  = embedment length

## About the authors



Bijan Khaleghi is the state bridge design engineer at the Bridge and Structures Office of the Washington State Department of Transportation (WSDOT) in Olympia, Wash.



Eric Schultz is a bridge designer at the Bridge and Structures Office of WSDOT in Olympia.



Stephen Seguirant is vice president and director of engineering at Concrete Technology Corporation in Tacoma, Wash.



Lee Marsh is a senior project manager at BergerABAM Inc. in Federal Way, Wash.



Olafur Haraldsson is a graduate student assistant in the Department of Civil Engineering at the University of Washington in Seattle.



Marc Eberhard is a professor in the Department of Civil Engineering at the University of Washington in Seattle.



John Stanton is a professor in the Department of Civil Engineering at the University of Washington in Seattle.

## Abstract

Prefabricated bridge components are in increasing demand for accelerated bridge construction. Precasting eliminates the need for forming, casting, and curing of concrete on site, making bridge construction safer while improving quality and durability. This paper describes the development and implementation of a precast concrete bridge bent system suitable for accelerated bridge construction in high seismic zones, such as western Washington. At the base of the bent, the column is connected to a spread footing using a socket connection, while at the top the column is joined to the cap beam using bars grouted in ducts. In both cases the connection was verified by testing before the system was implemented on-site by the Washington State Department of Transportation.

## Keywords

ABC, accelerated bridge construction, bridge, connection, LRFD, rapid construction, seismic.

## Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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