HOW TO BUILD A BRIDGE - FAST

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Abstract

A new connection has been developed for connecting precast bridge columns to cast-in-place spread footings. It is quick and simple to construct, and has excellent seismic resistance as well. It is made by precasting the column; setting, plumbing and leveling it on site; fixing the footing steel; and casting the concrete for the footing. No steel crosses the interface between the precast column and the spread footing; the column bars are all straight and are terminated with heads for anchorage. The column has a roughened surface that transfers the shear stresses (due to column axial load and bending) across the column-footing interface. Tests were carried out on the system. The paper describes the design criteria and methodology, the results of the tests, and recommendations for use of the system in practice.

Introduction

Bridge construction frequently leads to traffic delays, which result in wasted time and fuel. Bridge owners are therefore seeking methods to **a**ccelerate **b**ridge **c**onstruction, referred to as ABC. Such methods also offer reduced environmental impacts, better worker safety, higher quality construction and lower lifecycle costs (Wacker et al. 2005). Use of precast concrete represents a promising technology for ABC, and has been successfully used for bridge substructures in non-seismic regions (Matsumoto et al. 2001). 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 inelastic cyclic loads, but are also readily constructible, is challenging.

A bridge bent system has been developed at the University of Washington, in collaboration with WSDOT and Berger/ABAM Engineers, that is intended to satisfy the combined needs of seismic performance and rapid construction. Different connection systems are used at the column-to-foundation and the cap beam-to-column interfaces, because of the different conditions that exist at each location. The cap-beam connection is described in Pang et al. (2010) and is shown in Figure 1. It consists of bars that project from a precast column and are grouted into ducts in the precast cap beam. The distinction between it and other grouted bar systems is that large bars (up to #18 US, or D57) are used in large ducts. That approach allows the use of a small number of bars, which minimizes the number of bar fit-ups needed on site and maximizes the size of the ducts, both of which improve constructability.

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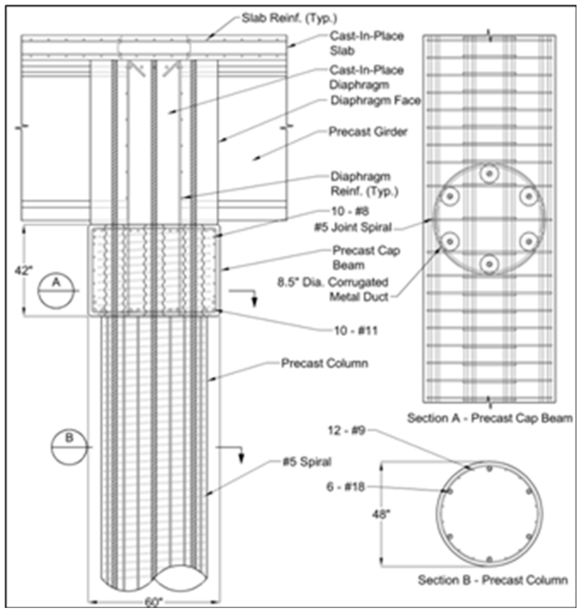


FIGURE 1. COLUMN TO CAP BEAM CONNECTION CONCEPT. (1 inch = 25 mm)

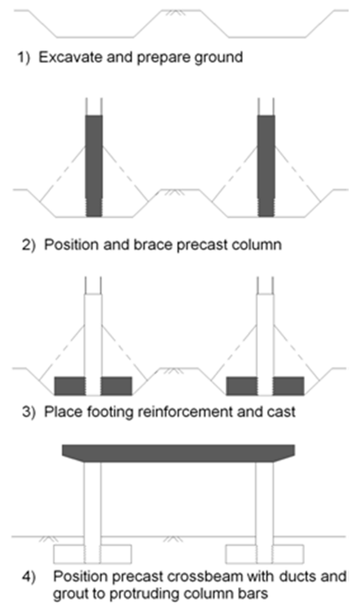


FIGURE 2. CONSTRUCTION SEQUENCE

The potential drawback of the use of large bars is their anchorage within the depth of the cap beam. However, extensive testing (Steuck et al. 2009) showed that even #18 (D57) bars could easily be anchored within the depth of a typical cap beam and that, under cyclic lateral load, the connection behaved like a cast-in-place system with conventional detailing and bar sizes (Pang et al. 2010).

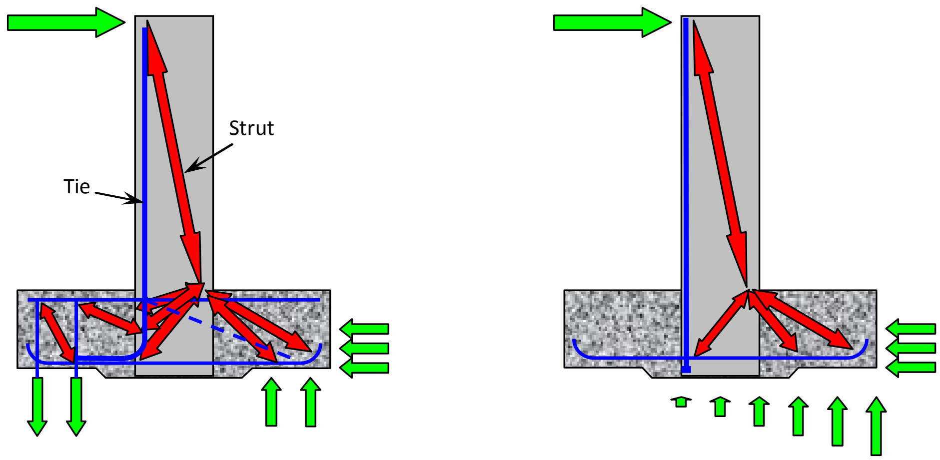
This paper describes the socket connection between the column and spread footing, for which the same requirements, namely simplicity of construction and good seismic performance, exist. Research to extend the concept to the use of drilled shafts is ongoing.

The construction sequence for the new socket connection is shown in Figure 2. The column is precast with a roughened outer surface at the bottom. Once the footing has been excavated (Step 1 in Figure 2), the precast column is brought to site, plumbed, leveled, and braced (Step 2). Footing reinforcement is then placed, and the footing is cast (Step 3) around the column.

The final step is to connect the column to the precast cap-beam (Step 4) by grouting the large bars in the large ducts. In comparison with conventional cast-in-place construction, the primary advantage of this system is construction speed; a footing and column can be built in little more time than is needed to cast the footing alone. Further, the use of a precast cap beam is estimated to save several weeks.

The structural details differ from those of a conventional, cast-in-place system in two ways. First, no bars pass from the footing into the column, so the only resistance to vertical load comes from shear friction across the precast to cast-in-place interface. That interface is intentionally roughened to facilitate this load transfer.

Second, the longitudinal column bars are not bent out at the bottom, but instead, they are developed by headed anchors. This choice simplifies transportation and handling of the columns, because no steel projects from the sides, and it eliminates the hazard that would otherwise be posed by protruding bars. The configuration also provides a much simpler and more direct flow of internal forces than is possible with bent-out bars. The distribution of internal forces is illustrated by the strut-and-tie model shown in Figure 3 for both a bent-out bar system and an anchored bar system. In the headed bar system, the node that connects the vertical column bar to the diagonal strut is a CCC node, which is extremely efficient, robust and reliable, because the concrete is in triaxial compression. In the conventional bent-out bar system, the load must be transferred from the diagonal strut to the bar by bond around the bend of the bar. This is a poor transfer mechanism and leads to diagonal cracks in the footing at relatively low stresses. These cracks are often referred to as “joint shear” cracks. The Caltrans Criteria (2006) require significant amounts of tie steel in the footing to overcome the poor transfer mechanism. That tie steel is not needed when headed bars are used.



(a) With bent out bars

(b) With headed bars

FIGURE 3. STRUT AND TIE MODELS OF CONNECTION.

This socket connection was used in a bridge over I-5 in Washington State that was constructed during the summer of 2011.

Design Requirements

The footing must satisfy several design requirements imposed by the AASHTO LRFD Design Specifications and the AASHTO Guide Specifications for LRFD Seismic design. The latter are largely based on the Caltrans Seismic Design Criteria (2006), but the Caltrans Criteria contain some requirements beyond the AASHTO ones, and they are expected to be incorporated into the next edition of the AASHTO

Guide Specifications. Therefore they were included too. The primary requirements are:

- The soil pressure under the footing, under vertical load plus overturning, must remain below the allowable bearing pressure.
- The resultant vertical reaction, under vertical load plus overturning, must remain within the middle two-thirds of the footing.
- The connection between the footing and column must provide sufficient strength to force the failure to occur in the column. The goal is to avoid footing failure, which can be expensive to inspect and repair.

The first two requirements, based on soil properties, essentially define the plan dimensions of the footing and are not discussed further here. The structural requirements for the connection can be identified from the potential modes of failure. The primary ones are:

- Bending strength of the footing.
- One-way shear strength of the footing.
- Anchorage failure of the bars.
- Punching shear failure under vertical load (assumed to occur on a conical surface).
- Transfer of combined vertical load and moment between the column and footing.
- Shear friction failure across the precast to cast-in-place interface.
- Joint-shear failure within the connection.

The longitudinal column bars are equipped with headed anchors, which should be selected from commercially available products and may thus be assumed to provide adequate anchorage. The other structural requirements depend on the flow of forces in a system that is highly statically indeterminate. While the distribution of internal forces is expected to follow the general pattern shown in Figure 3, the resistance in each potential failure mode is not well defined and so needs to be determined by testing. The characteristics that were expected to influence the resistance were:

- The ratio of footing depth to column diameter.
- The quantity of column longitudinal reinforcement.
- The quantity of shear reinforcement in the column within the joint region.
- The quantity of transverse reinforcement in the footing.

Test Program

The number of characteristics exceeded the number of tests (three) that could be conducted within the program resources, so choices had to be made. All the test specimens consisted of cantilever columns projecting from spread footings, in which the columns were subjected to constant vertical load and cyclic lateral load. The test specimens consisted of 20-in. (500 mm) diameter columns and represented at 5/12 scale a notional prototype with a 4-ft (1200 mm) diameter column. Specimens SF-1 and SF-2 each contained a column splice above the plastic hinge region. However, the splice is not an essential part of the system, and is not discussed further here.

In many columns in the field, the longitudinal column reinforcement ratio is close to the AASHTO minimum of 1%. Therefore this ratio was used for all three tests. The spiral steel in the column is also typically carried down into the footing at the same pitch as in the body of the column, so that was also done in all of these test specimens. It provided joint shear some resistance.

The primary test variables were the ratio of footing thickness to column diameter and the quantity of transverse steel used in the footings. In addition several other details of the footing steel were varied between specimens. The specimen details are summarized in Table 1. The test program was carried out while the Washington State Department of Transportation, WSDOT, was designing a bridge that uses the system. The bridge construction started only eight months after that laboratory testing was complete, and is now finished. The geometry of the test specimens represented, at 1: 2.4 scale, the geometry of the prototype bridge components.

TABLE 1. TEST SPECIMEN DETAILS

Spec. No. (-)	Column dia. (in)	Vertical rft ratio (%)	Spiral rft ratio (%)	Footing depth (in)	Footing Ties (-)	Diagonal steel sets (-)
SF-1	20	1.12	0.88	22	full	3
SF-2	20	1.12	0.88	22	half	1
SF-3	20	1.12	0.88	10	none	0

(Note: 1 inch = 25 mm).

Specimen SF-1 was regarded as the most conservative detailing, intended to represent as closely as possible a direct conversion from cast-in-place to precast construction. Thus some of the flexural steel in the bottom of the footing passed directly under the column, and this required casting a slot into the bottom of the column, as shown in Figure 4. The goal was to ensure the best possible engagement of the footing steel with the column steel. A plan view of the test specimen is also shown in Figure 5.

In all three specimens, the column projected slightly below the structural part of the footing in order to locate the anchor head on the column bar just below the bottom node in the strut and tie model shown in Figure 3. This choice would cause the node to behave as a CCC node, with the attendant stable load transfer properties and absence of anchorage problems. A void was also left between the underside of the column and the top of the test floor to ensure that all of the applied vertical load was resisted by the connection between the column and footing, because none could pass in bearing to the platen of the test machine under the footing.

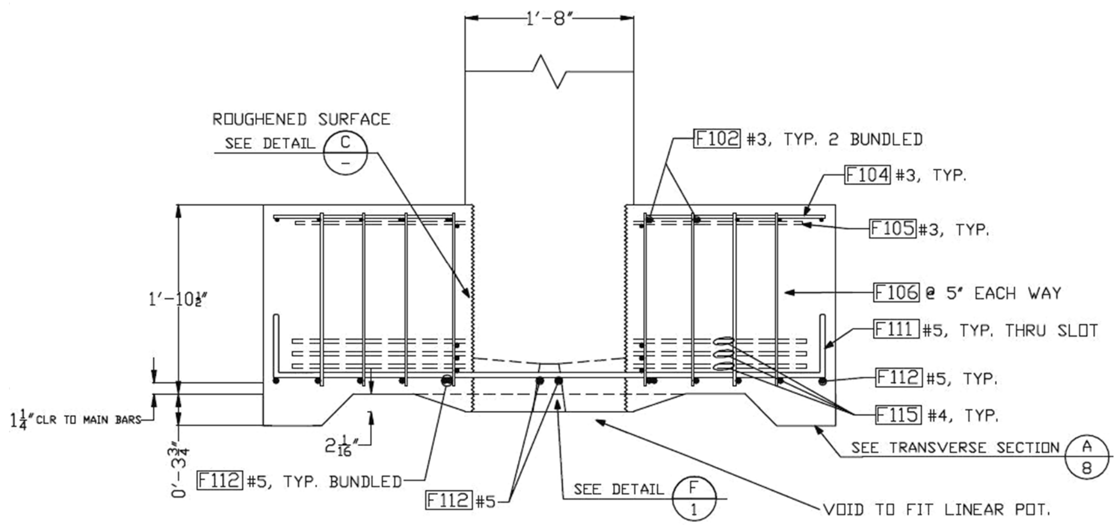


FIGURE 4. SPECMEN SF-1: SECTION. (1 inch = 25 mm)

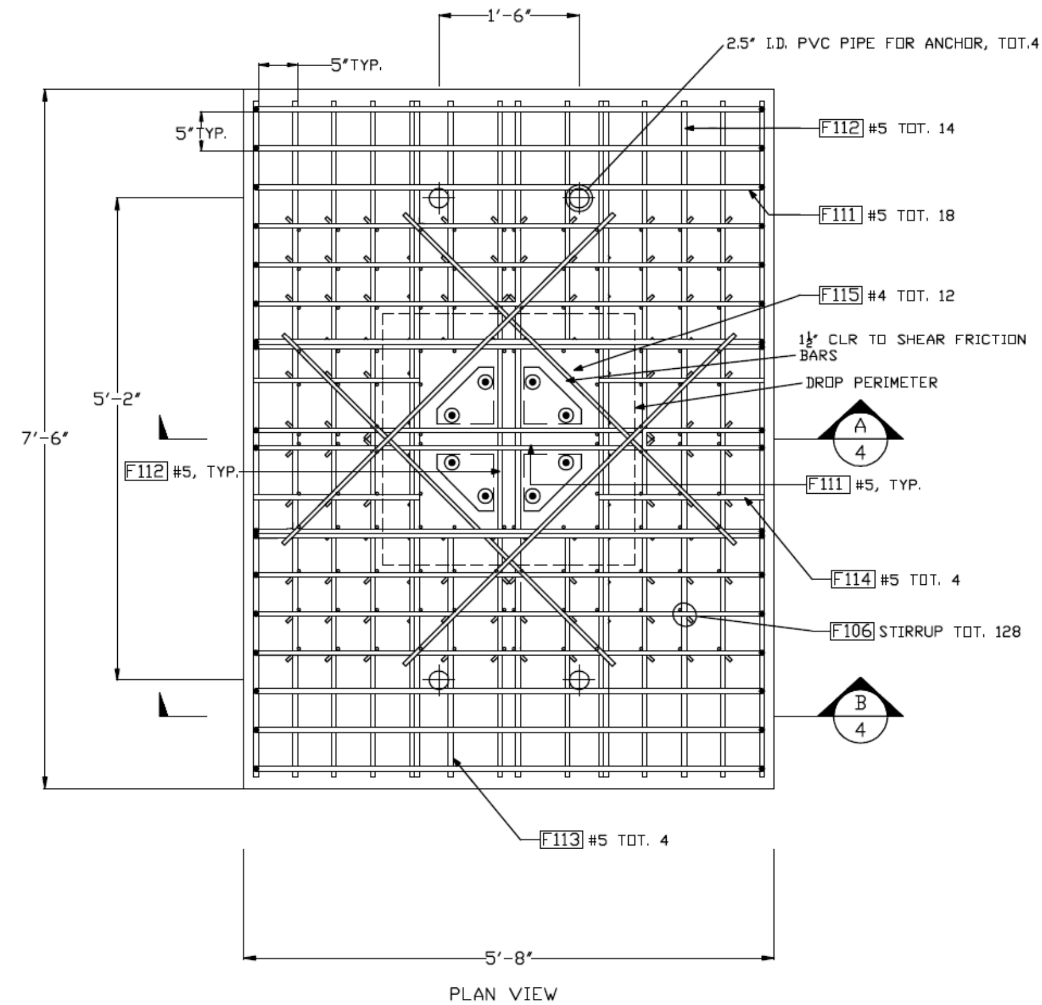


FIGURE 5. SPECIMEN SF-1: PLAN VIEW. (1 inch = 25 mm).

Figure 5 also shows sets of diagonal bars in the footing. These were placed both to provide some reinforcement in the otherwise unreinforced corners of the square region of cast-in-place concrete surrounding the octagonal column, and to provide a tension capacity across the pc-cip interface for the purpose of generating shear friction resistance there. Last, the footing of Specimen SF-1 contained the full complement of transverse reinforcement required by the Caltrans Criteria (2006). That reinforcement is intended to resist joint shear forces. It should be noted that, unlike a beam-column joint in a building frame, the region that constitutes the joint, and in which the reinforcement can be placed, includes a region of the footing outside the column itself.

Specimen SF-2 was similar to SF-1 except that no footing steel was placed under the column, thereby eliminating the slots in the bottom of the column. The same total amount of footing steel was used, but some bars were moved to just outside the column where they were bundled with other bars already in that location. The diagonal shear friction steel was also reduced to a single set of bars, rather than three sets, and the amount of transverse steel in the footing was halved. The basis for reducing the diagonal steel was that, in both specimens, the normal force due to the flexural steel was ignored in evaluating the shear friction resistance, even though it appears logical to count it. (This is evident from the strut and tie model in Figure 3b). The transverse steel was reduced because the tests underlying the Caltrans Requirements were all conducted on cast-in-place systems in which the column bars were bent out. That arrangement prevents formation of the strut and tie model of Figure 3b, in which case the forces must follow a more complex path, such as that in Figure 3a (from Xiao et al. 1996). It was hypothesized that replacement of the bent-out bars by anchor heads in the present study would reduce or eliminate the need for additional transverse reinforcement.

Specimen SF-3 was designed and constructed after testing SF-1 and SF-2. Because those two suffered essentially no damage in the connection region, they provided only lower bounds on the connection strength. To obtain an upper bound as well (and therefore to bracket the true value), failure in Specimen SF-3 had to be forced into the connection region. To do that, it was designed with a 20-in. (500 mm) diameter column, as in Specimens SF-1 and SF-2, but a footing that was only 10 in. (250 mm) thick. The column steel remained the same, but the flexural steel in the footing had to be much heavier to provide the same flexural strength with a smaller lever arm.

To avoid a spurious one-way ("beam") shear failure, a small number of footing ties were needed for one-way shear strength. They were placed so that they contributed to one-way shear resistance but not to punching shear resistance. A single set of diagonal "shear friction" steel bars was used, as was done in Specimen SF-2, to provide a minimum of reinforcement in the otherwise reinforced corners of the region around the column. The results of tests on Specimens SF-1 and SF-2 had shown that the stress in the diagonal bars never exceeded about 5% of yield, so they were not expected to contribute significantly to shear friction resistance in Specimen SF-3.

Test Results

Each test specimen was first subjected to a pure axial load test to investigate the possibility of shear failure at the precast-cast-in-place interface. The axial load for Specimens SF-1 and SF-2 was 240 kips (1077 kN), which was the factored DL + LL on the prototype bridge to be built over I-5, scaled to specimen size. Specimen SF-3 was subjected to 1.4 times this load. No signs of cracking or damage were seen in any of the three specimens under this loading.

Each test specimen was then subjected to the lateral displacement history shown in Figure 6. Displacements were applied under stroke control.

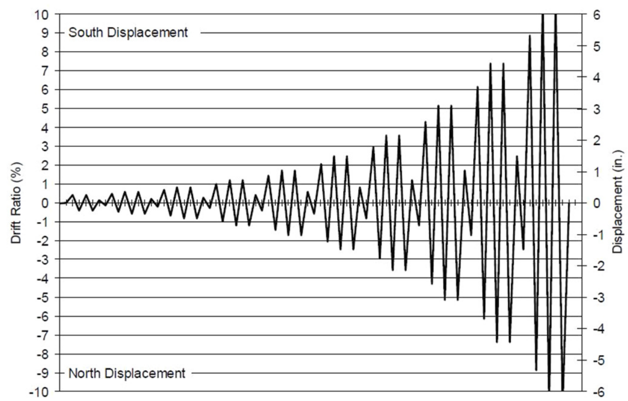


FIGURE 6. LATERAL DISPLACEMENT HISTORY.

The load-displacement plots for all three specimens are shown in Figure 7.

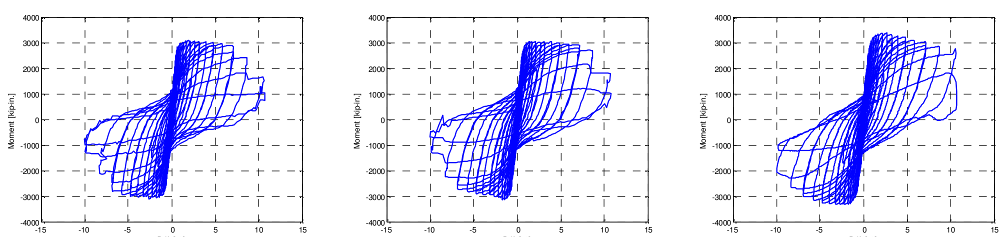


FIGURE 7. LOAD-DISPLACEMENT RESPONSE FOR SPECIMENS SF-1, SF-2 AND SF-3. (1 inch-kip = 113 N-m).

The responses of Specimens SF-1 and SF-2 were nearly identical, and furthermore they were essentially the same as that of a cip reference specimen tested earlier (Pang et al. 2010). All the damage occurred in the column, which failed by combined axial load and flexure in a conventional plastic hinge. The proximate cause of failure was fracture of some of the bars, which was in turn caused by buckling and re-straightening of those bars under cyclic loading. Bar buckling was first observed at approximately 6% drift ratio in both cases, after which the lateral strength started to drop. The only cracks in the footing were hairline, and all the footing steel displayed stresses well below yield. The footing was thus behaving as if it were made from mass concrete with no reinforcement.

Specimen SF-3, with the thinner footing, behaved differently. In the early stages of loading it appeared to be behaving in the same way as the other two, with some spalling of the column concrete and yielding of the longitudinal column steel. However, at about 6% drift, some spalling of the footing concrete became visible around the column. The column bars did not buckle or fracture, but the footing started to sustain more damage. Eventually, at 10% drift, the specimen failed by combined vertical force and moment transfer in the connection region. Because a void had been deliberately left under the column to prevent vertical support from the test floor, the column sank approximately 3 in. (75 mm) through the footing when failure occurred. Thus the objective of forcing failure to occur in the footing was achieved. However, the extensive yielding of the column bars and the fact that no footing damage was visible until quite late in the loading history suggest that the specimen was only just connection-critical, in which case the ratio of column diameter to footing thickness used in the specimen represents a fairly tight upper bound on the value needed to avoid footing failure.

Figure 8 shows Specimen SF-3, seen from below, after failure. The column can be seen to be relatively intact, with all the damage concentrated in the surrounding footing concrete. The failure surface suggests a punching shear failure under combined vertical load and bending.

FIGURE 8. SPECEIMEN SF-3 AFTER FAILURE (FROM BELOW).

After the lateral-load testing, Specimens SF-1 and SF-2 were subjected to a vertical load test to failure, in order to evaluate the remaining strength of the connection under vertical load alone. The test could not be applied to Specimen SF-3 because the connection region in it had already failed. In both cases, the column was able to carry approximately 840 kips (3740 kN) before crushing in the plastic hinge region of the column. It should be noted that the spiral and several longitudinal bars had already fractured in the lateral load testing, before this vertical load was applied. No damage occurred in the footings during this loading. The peak load of 840 kips (3740 kN) was limited by the axial strength of the previously-damaged column, so the footing capacity may have been much higher. The load represents 3.5 times the factored dead plus live load in the prototype bridge, adjusted to laboratory scale. It is thus clear that for footings of these proportions, the shear friction capacity across the pc-cip interface is easily sufficient to resist the vertical load.

Conclusions

The following conclusions were drawn from the study:

- **Connection concept.** The column-to-footing socket connection can be designed so that the system behaves like a comparable cast-in-place column-to-footing connection. The precast columns can be designed following the same specifications as are used to design conventional cast-in-place columns.
- **Need for mechanical anchors.** The use in the column of headed straight bars, instead of bent out bars, simplifies construction and improves the force flow in the footing, but necessitates the use of headed anchors on the bars.
- **Design against footing failure.** The procedures outlined in the AASHTO Guide Specifications for LRFD Seismic Design for determining the required flexural strength of the footing were effective in preventing footing failure in all three column tests.
- **Vertical ties in the footing.** When the column steel consists of straight bars equipped with headed anchors, rather than the conventional bent-out bars, the prescriptive vertical footing ties specified by the AASHTO Guide Specification perform no useful function and can be omitted. This conclusion applies only to the prescriptive ties, and not to ties that are needed to supply shear resistance required to resist computed shear demands.
- **Shear-friction push-through resistance of connection.** The strength of the connection in shear friction was sufficient to prevent any sign of slip, much less sliding failure, across the pc-cip interface, in any of the three test specimens. The flexural reinforcement provides normal forces across the potential sliding interface, which induce sufficient friction to resist the demands. Thus additional reinforcement (here placed diagonally) is not necessary for that purpose. However, a small amount of diagonal "trimming" reinforcement is still desirable to avoid the existence of a large region of unreinforced concrete at the corners of the embedded column.

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