Seismic Performance of Precast Concrete Bents used for Accelerated Bridge Construction

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Abstract

Ductility of precast prestressed girder bridges can be achieved by proper detailing of pier diaphragm through extended strands, column bars, and joint reinforcement. Extended strands at intermediate crossbeams are used to connect the ends of girders with diaphragms and resist loads from creep effects, shrinkage effects, and seismic positive moments. 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. In both cases the connection was verified by testing before the system was implemented. This paper describes the development, experimental validation, and implementation of a precast concrete bridge bent system that is intended to meet those challenges. A precast concrete bridge bent system is presented that is conceptually simple, can be constructed rapidly, and offers excellent seismic performance.

Introduction

Seismic design of precast concrete bridges begins with a global analysis of the response of the structure to earthquake loadings and a detailed evaluation of connections between precast girders and connections between the superstructure and the supporting substructure. Ductile behavior is desirable under earthquake loadings for both the longitudinal and transverse directions of the bridge. Further, the substructure must be made to either protect the superstructure from force effects due to ground motions through fusing or plastic hinging, or to transmit the inertial forces that act upon the bridge to the ground through a continuous load path.

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.

Performance Criteria For Prestressed Girder Ordinary Bridges

Designing for life safety means that significant damage can result. Significant damage includes permanent offsets and damage between approach structures and the bridge superstructure, and between spans at expansion joints, permanent changes in bridge span lengths, and permanent the basis of their stiffness distribution factors. This moment

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displacements at the top of bridge columns. Damage also consists of severe concrete cracking, reinforcement yielding and buckling, major spalling of concrete and severe cracking of the bridge deck slab. These conditions may require closure of the bridge to repair the damages. Partial or complete replacement of columns may be required in some cases. For sites with lateral flow due to liquefaction, piles and shafts may suffer significant inelastic deformation, and consequently, partial or complete replacement of the columns, piles and shafts may be necessary. If replacement of columns or other components is to be avoided, a design strategy that produces minimal or moderate damage, such as seismic isolation or a control and reparability design concept, should be used. Figure 1 shows the connection concept is commonly used by WSDOT for bridges in moderate and high seismic zones.

Girders and deck slab are continuous at piers with girders framed into the pier diaphragm. Such structures are thought to exhibit behavior as a continuous superstructure with a fixed moment resistant connection to the substructure.

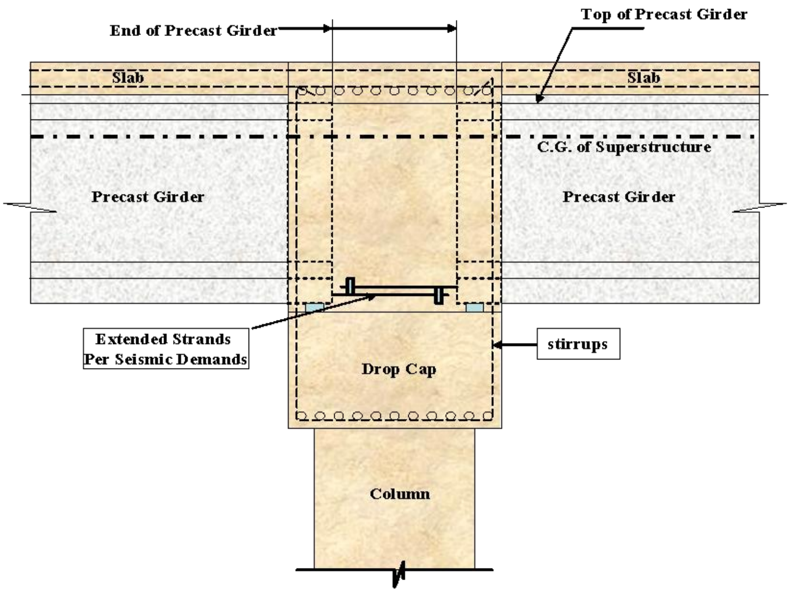


Figure 1: Pier with Girders Framed into the Pier Diaphragm

Plastic hinges form before any other failure due to overstress or instability in either the overall structure, or in the foundation, or both. Plastic hinges are permitted only at locations in columns where they can be readily inspected and repaired. Superstructure and substructure components and their connections to columns that are not designed to yield are rather designed to resist overstrength moments and shears of ductile columns. The plastic moment capacity for reinforced concrete columns is determined using a moment curvature section analysis, taking into account the expected yield strength of the materials, the confined concrete properties, and the strain-hardening effects of the longitudinal reinforcement.

Capacity-protected members such as bent caps, joints at top and bottom of column, and integral superstructure elements that are adjacent to the plastic hinge locations are designed to remain essentially elastic when the plastic hinge reaches its overstrength moment capacity. The superstructure is designed as a capacity protected member. Any moment demand caused by dead load or secondary prestress effects in case of continuous tendons over piers is distributed to the entire width of the superstructure. The column overstrength moment, in addition to the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the superstructure, is distributed to the spans framing into the bent on demand is considered within the effective width of the superstructure.

Positive Moment Connection At Pier Diaphragms

The procedure used to calculate the required number of extended strands is described in this section. Calculations assume the development of the tensile strength of the strands at ultimate loads. Strands used for this purpose must be developed within the short distance between the two girder ends.2

The design moment at the center of gravity of the superstructure, Mpo CG is calculated using the following equation:

$$M_{po}^{CG} = M_{po}^{top} + \frac{\left(M_{po}^{top} + M_{po}^{Base}\right)}{L_c} h \qquad (1)$$

Where:

M_{po}^{top} = plastic overstrength moment at top of column, ft-kips

M_{po}^{Base} = plastic overstrength moment at base of column, ft-kips

h = distance from top of column to c.g. of superstructure, ft

Lc = column clear height used to determine overstrength shear associated with the overstrength moments, ft

This moment is resisted by the bent cap through torsion. The torsional capacity of the bent cat shall be investigated. The torsion in the bent cap is distributed into the superstructure based on the relative flexibility of the superstructure and the bent cap. Hence, the superstructure does not resist column overstrength moments uniformly across its width. To account for this, an effective width approximation is used, where the maximum resistance per unit of superstructure width of the actual structure is distributed over an equivalent effective width to provide an equivalent resistance[3]. The equivalent width concept is illustrated in Figure 2.

For concrete bridges, with the exception of box girders and solid superstructures, this effective width can be calculated as follows:

$$Beff = D_c + D_s \qquad (2)$$

Where:

D_c diameter of column

D_s depth of superstructure including cap beam

Total number of extended straight strands, Nps, needed to develop the required moment capacity at the end of girder is based on the yield strength of the strands:

$$N_{ps} = 12\left[M_{sei} \bullet K - M_{SIDL}\right] \bullet \frac{1}{0.9\phi A_{ps} f_{py} d} \qquad (3)$$

where:

A_{ps} area of each extended strand, in.2

f_{py} yield strength of prestressing steel, ksi

d	distance from top of slab to c.g. of extended strands, in.
MSIDL	moment due to SIDL (traffic barrier, sidewalk, etc.) per girder
K	span moment distribution factor
ϕ	strength reduction factor for flexure

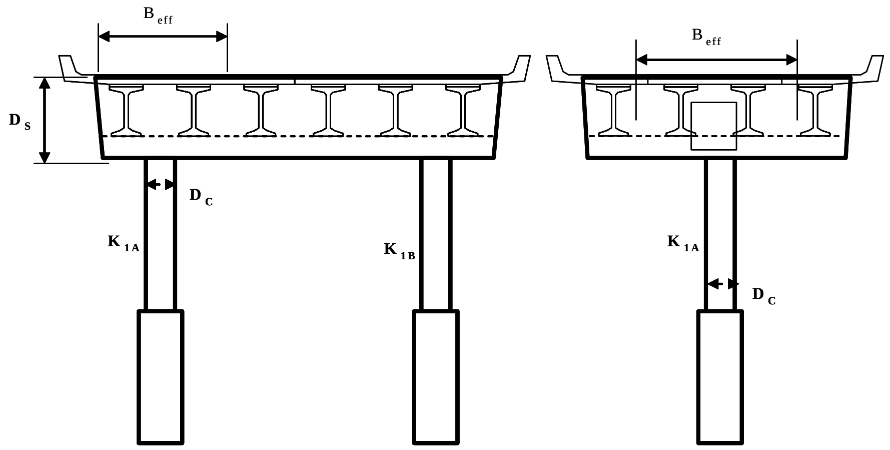
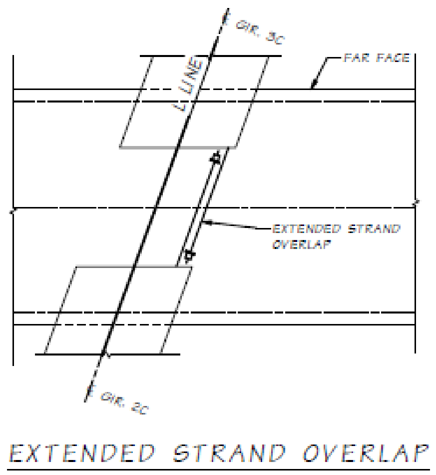


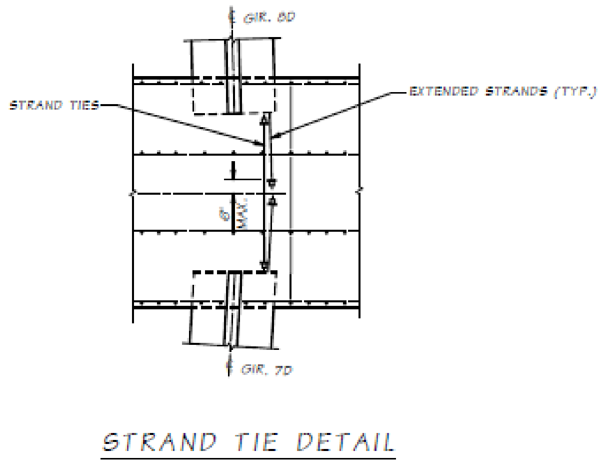
Figure 2: Effective Superstructure Width for Extended Strand Design

Continuity Of Extended Strands

Continuity of extended strands is essential for all prestressed girder bridges with fixed diaphragms at piers. Strand continuity may be achieved by directly overlapping extended strands as shown in Figure 3a, by use of strand ties as shown in Figure 3b in case of curved superstructures with corded precast girders, by the use of the crossbeam ties as shown in Figure 4 along with strand ties, or by a combination of all three methods.



3a: Overlapping Extended Strand



3b: Strand Ties

Figure 3: Overlapping Extended Strand and Strand Ties

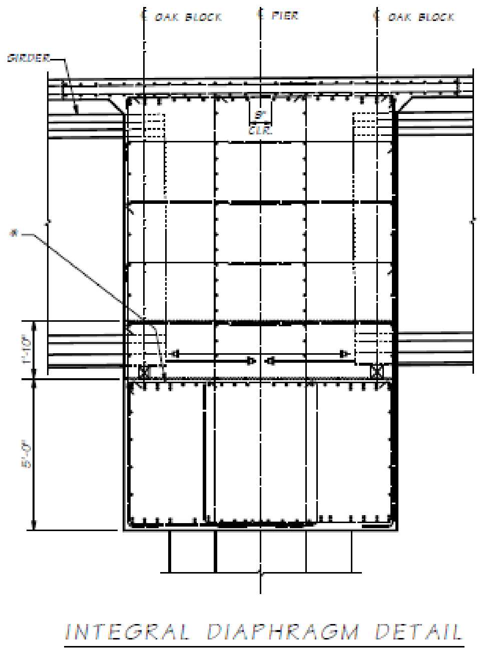


Figure 4: Lower Crossbeam Ties

Strand ties are used at piers with a girder angle point due to horizontal curvature where extended strands are not parallel and would cross during girder placement. The area of transverse ties considered effective for strand ties development in lower crossbeam should not exceed:

$$A_s = \frac{1}{2}\frac{A_{ps} f_{py} n_s}{f_{ye}} \qquad (4)$$

Where:

A_{ps}	Area of strand ties, in2
n_s	Number of extended strands that are spliced with strand and crossbeam ties
f_{py}	Yield strength of extended strands, ksi
f_{ye}	Expected yield strength of reinforcement, ksi

The above equation is driven from the strut and tie model considering the 3-dimensional effect and conservative engineering judgment. Two-thirds of As is placed directly below the girder and the remaining part of As is placed outside the bottom flange width. The size of strand ties is the same as the extended strands, and is placed at the same level and proximity of the extended strands.

Joint Performance For SDCs C And D

Moment-resisting connections for prestressed girder bridges in SDCs C and D are designed to transmit the maximum forces produced when the column has reached its overstrength capacity. A "rational" design is required for joint reinforcement when principal tension stress levels become excessive. The amounts of reinforcement required are based on a strut and tie mechanism similar to that shown in Figure 5 for a hammerhead pier crossbeam[4].

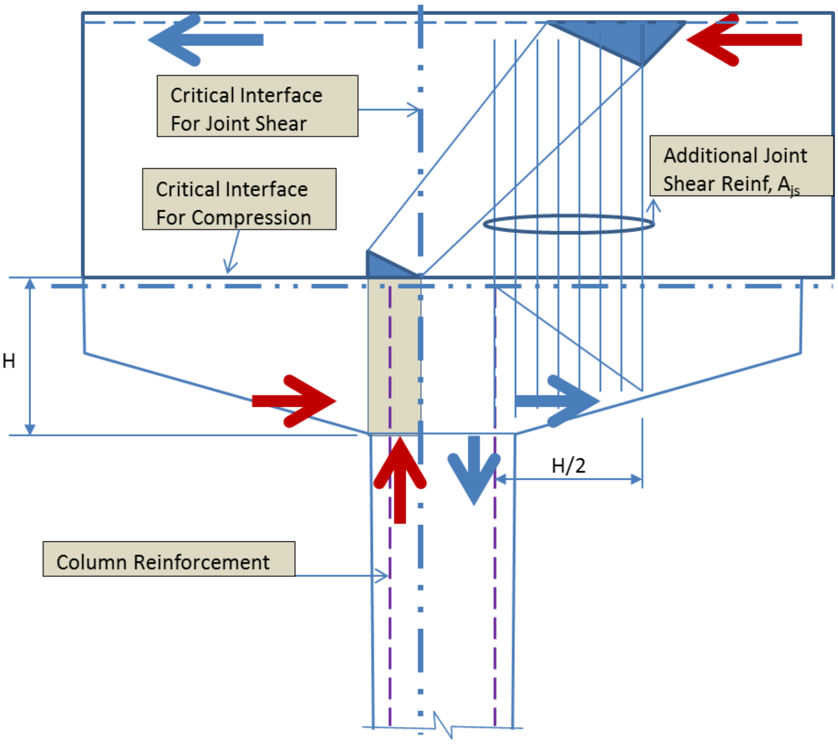


Figure 5: Strut and Tie Model of Intermediate Diaphragm

For precast prestressed girder bridges in SDCs C and D with fixed diaphragms at piers, all column longitudinal reinforcement should be extended into the cast-in-place concrete diaphragm on top of the crossbeam. For bridges in SDC B with fixed diaphragms at piers, column longitudinal reinforcement can be terminated at top of lower crossbeam. Column longitudinal reinforcement can be terminated at top of lower crossbeam in all SDCs if analysis shows that plastic hinging will not occur at the top of column under the design earthquake.

In case of interference, column longitudinal reinforcement obstructing the extended strands should be terminated at the top of the lower crossbeam, and should be replaced with the equivalent full height stirrups extending from the lower to upper crossbeam within the effective zone. The effective zone is defined as the width Dc+Ds1 for columns without headed bars and Dc+2 Ds1 for columns with headed bars, where Dc is the column width or diameter, and Ds1 is the depth of lower crossbeam. Headed bars are only used if the depth of the lower crossbeam is less than 1.25 times the tension development length of column longitudinal reinforcement.

ASTM A706 Grade 80 reinforcing steel may be used for capacity-protected members such as footings, bent caps, oversized shafts, joints, and integral superstructure elements that are adjacent to the plastic hinge locations if the expected nominal moment capacity is determined by strength design based on the expected concrete compressive strength with a maximum usable strain of 0.003, and a reinforcing steel yield strength of 80 ksi with a maximum usable strain of 0.090 for #10 bars and smaller, and 0.060 for #11 bars and larger. The resistance factors for seismic related calculations are taken as 0.90 for shear and 1.0 for bending. ASTM A706 Grade 80 reinforcing steel should not be used for transverse reinforcement in members resisting torsion. The applicability of AASHTO SGS is limited to grade 60 ksi reinforcing steel. Since the suitability of ASTM A706 Grade 80 ksi reinforcing bars for ductility and confinement has not been tested, the applicability should be limited only to non-ductile elements.

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 Specifications1 and displacement-based design by the AASHTO Guide Specification for LRFD Seismic Bridge Design.

WSDOT's seismic design is based on the AASHTO guide specification modified by the WSDOT Bridge Design Manual. 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[4].

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.
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 stiffness 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.

Implementation – From Research To Practice

Figure 6 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[5].

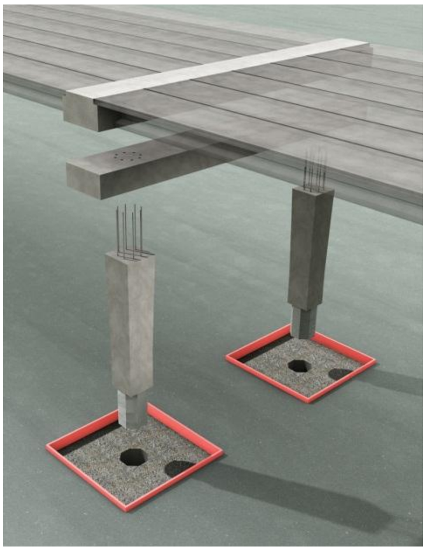


Figure 6. Precast concrete bent system configuration.

Figure 7. Previous use of precast concrete cap beam that used in Washington State.

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. The bridge site is in a congested 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.

The material characteristics in the tests included ASTM A70610 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 8 summarizes the results of the pullout tests. It shows the bar stress at failure plotted against the ratio of embedment length to bar diameter le/dbto 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/yd3 (1.8 kg/m3). They were used in some pull- out 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 le/db. 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 6db and 10db, respectively.

Once the anchorage properties under monotonic tension loading had been established, column-to-cap beam connection tests were conducted under cyclic lateral loading. Figure 10 shows a typical test. The 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[6].

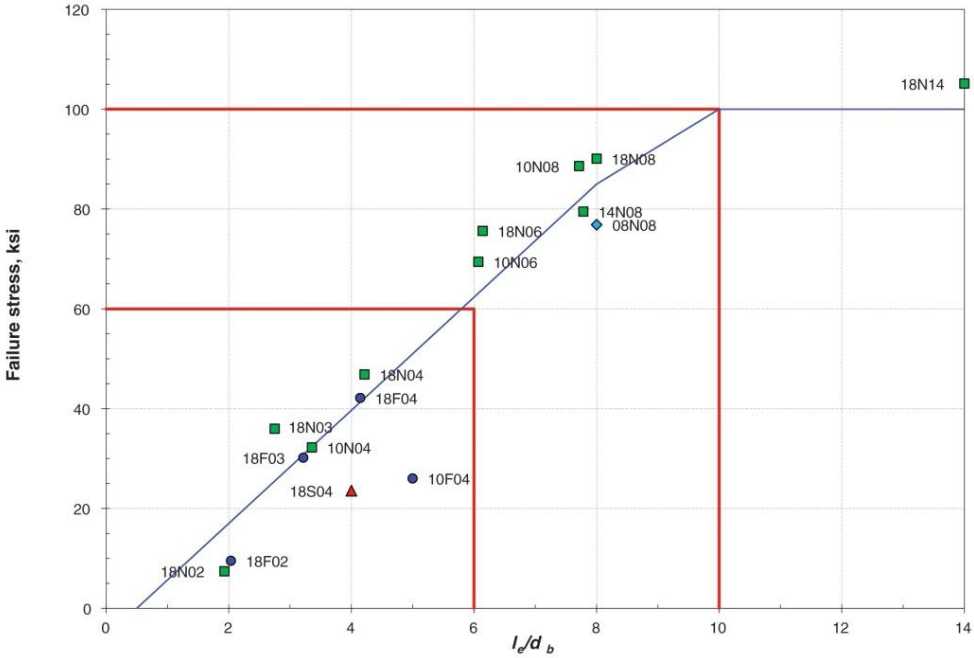


Figure 8. Grouted bar-duct pullout test results. Note: db = bar diameter; le = embedment length. 1 ksi = 6.895 MPa.

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

Figure 9. Construction and testing of precast concrete column-to-footing connection.

Site implementations- From Research to Practice

Figures 10 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

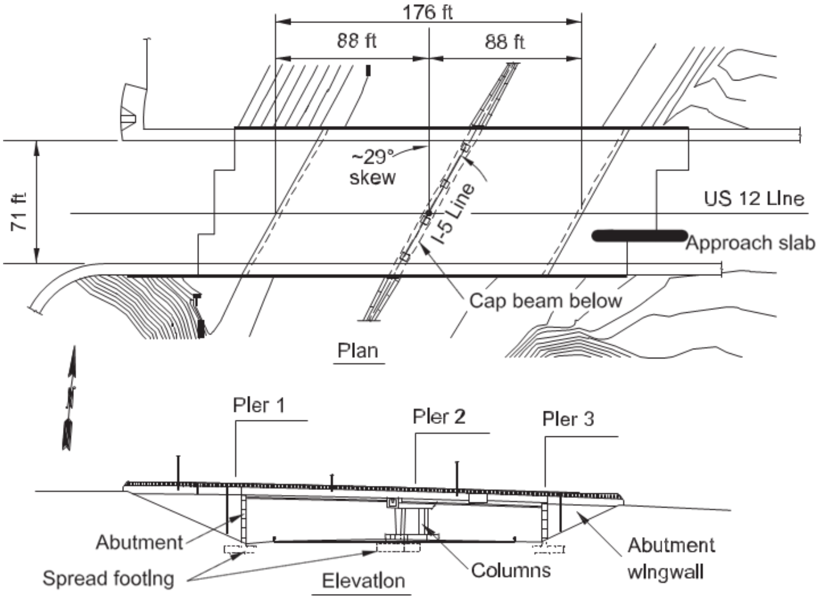


Figure 10. Bridge layout for demonstration project.

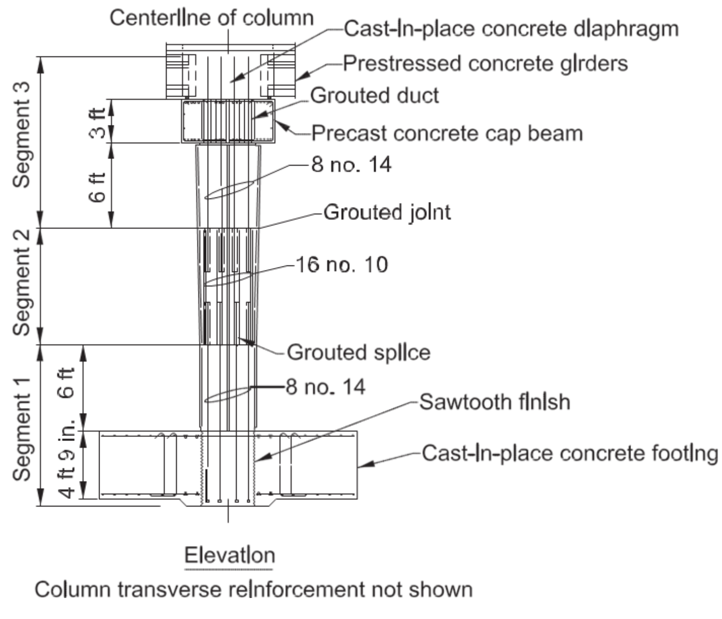


Figure 11. Demonstration bridge column details for elevation.

The construction sequence for placement of the precast concrete superstructure at the intermediate pier 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.

Figure 12. Placement of precast concrete cap beam.

Conclusion

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

1. 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.
2. 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.
3. 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.
4. 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.
5. Column longitudinal reinforcement can be terminated at the top of the lower crossbeam in all SDCs if analysis shows that plastic hinging will not occur at the top of the column under the design earthquake.

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