HIGHWAYS FOR LIFE PROJECTS AND ACCELERATED BRIDGE CONSTRUCTION IN WASHINGTON STATE

ABSTRACT

The Federal Highway Administration, as part of the “Every Day Counts” initiative, is actively promoting accelerated bridge construction (ABC) in an effort to reduce the construction time while improving work-zone safety and minimizing the environmental impacts. The “Every Day Counts” initiative promotes Highways for Life (HFL) projects which allow states to implement the new and innovative technologies for better performance of prefabricated bridge elements in seismic zones. Prefabricated bridge components are in increasing demand for accelerated bridge construction. Precasting eliminates the need for forming, casting, and curing of concrete in work zones, making bridge construction safer while improving quality and durability.

In prefabricated concrete systems, connections are often made by grouting bars that project from one member into ducts embedded in another. For bridge bents, bar-duct systems can be assembled rapidly if a few large bars and ducts are used to connect the column and cap beam. Lateral load tests on precast bent connections have shown that they have strength and ductility similar to those of a comparable cast-in-place connection.

This paper describes the Washington State Department of Transportation’s adoption of ABC and the ongoing HFL project using precast concrete bridge bent connections that are suitable for high seismic zones.

Keywords: Bridge, ABC, LRFD, HFL, Precast, Seismic, connections

INTRODUCTION

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. Precast concrete bridge systems provide effective and economical design solutions for new bridge construction as well as for the rehabilitation of existing bridges. The proper seismic design entails a detailed evaluation of the connections between precast components, as well as the connection between superstructure and the supporting substructure system. In seismic regions, provisions must be made to transfer greater forces through connections and to ensure ductile behavior in both longitudinal and transverse directions.

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. The use of precast concrete for bridge substructures in seismic regions represents promising technology for ABC. Precast connections are typically made at the beam-column and column-foundation interfaces to facilitate fabrication and transportation. For structures in seismic regions, however, those interfaces represent the locations of high moments and large inelastic cyclic strain reversals. Developing connections capable in resisting cyclic loads, but also readily constructible, is the primary challenge for ABC in seismic regions. The precast concrete bridge bent system described in this paper is intended to meet those challenges.

Safety enhancements that benefit the motoring public and highway workers, as well as reduce environmental impacts, are directly attributable to limiting in-situ work requirements. For these reasons, transportation agencies are gradually embracing the ABC philosophy for many of their urban construction projects.

ACCELERATED BRIDGE CONSTRUCTION AND POTENTIAL PAYOFF

ABC can result in substantial economic benefits that can offset most construction cost premiums. Conventional bridge construction typically induces traffic delays and congestion for an extended time period. The induced traffic congestion adversely affects individual traveler's budgets and the region's economy, impacts air quality due to increased vehicle emissions, and reduces the quality of life due to personal time delays. Additionally, untimely service for the workforce, suppliers, and customers can incur significant costs to the traveling public and regional businesses.

Prefabrication of elements is the essence of accelerated construction. Although prefabrication can reduce total contract time, the critical issue is the time spent on site, because that determines the extent of the interruption to traffic, the fuel wasted by delays, and the excess of carbon dioxide which is likely to play a major role in the total cost of the project. The details of the ABC in WA and WSDOT strategic plan[4] for ABC are provided in reference 4.

Precast units are often constructed in specialized plants. There, repetitive construction permits investment in high-quality steel forms, which give rise to high-quality finishes and accurate dimensional control. Plant operations also allow tight quality control of materials, the possibility of prestressing, rapid production and good schedule control. Some of these advantages are also available with site precasting, which allows workers to work at ground level, and which provides the added advantage of removing the need for, and limitations of, long-distance transportation to the site.

WSDOT, as part of the ABC strategic plan, has developed a decision-making checklist to be considered at the project development stage to identify the suitability of a bridge project for ABC. The typical components of the decision-making matrix include high traffic volume, emergency replacement, worker safety concerns, high daily traffic control costs, evacuation route or over railroad or navigable channel, lane closures or detours, critical path of project, closure during off-peak traffic, rapid recovery/repair requirement, adverse economic impact, weather constraints, environmentally sensitive site, natural or endangered species, feasibility if bridge is historic, multiple similar spans, delay-related user cost concern, innovative contracting strategies, group with other bridges, and future use.

SEISMIC RESPONSE OF BRIDGES WITH PRECAST COMPONENTS

The most common types of connections for precast prestressed girder bridges are a fixed connection for high seismic zones (western Washington), and hinge connection for low seismic zones (eastern Washington). Precast concrete substructures can be used if monolithic moment resistant connections that meet seismic design and detailing requirements are provided.

Monolithic action between the superstructure and substructure components is the key to seismic resistant precast concrete bridge systems. Lack of monolithic action causes the column tops to behave as pin connections, resulting in substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismicity. Developing a moment connection between the superstructure and substructure reduces the moment in the footing. Figure 1 shows the moment diagram of a bridge pier with fixed connections subjected to seismic lateral load, with the maximum flexural moments occurring on the top and bottom of columns.

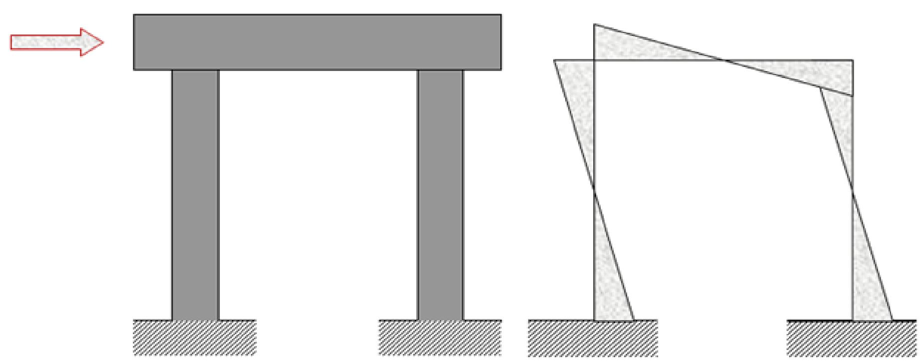


Figure 1: Moment Diagram of a Bridge Pier with Fixed Connections

The lack of monolithic action between the superstructure and bent cap in precast, prestressed concrete beam systems causes either the girder seats or the column tops to act as pinned connections. Consequently, while the transverse stability of multi-column bents is ensured by frame action in that direction, stability in the longitudinal direction requires the column bases to be fixed to the foundation supports. This requirement places substantial force demands on the foundations of multi-column bents, particularly in areas of moderate to high seismic zones. Developing a moment resisting connection between the superstructure and substructure makes it possible to develop plastic hinging at the column bases. This results in less expensive foundations. Integral bent caps are beneficial in precast, prestressed concrete beam systems by introducing moment continuity at the connection between the superstructure and the cap, which forces the columns into double-curvature bending and tends to substantially reduce their moment demands. As a result, the sizes and overall cost of the adjoining foundations are also reduced.

DESIGN SPECIFICATIONS AND GUIDELINES

Currently there are two methods for seismic design of bridges: 1) force-based design of the AASHTO LRFD Bridge Design Specifications[1] and 2) displacement-based design of the AASHTO Guide Specification for LRFD Seismic Bridge Design[2].

The provisions contained in the AASHTO LRFD Bridge Design Specifications are largely based on the Conventional Force method, where bridge analysis is performed and the forces on its various components are determined. Plastic hinging is the basis of ductile design for bridge structures. Plastic hinges may be formed at one or both ends of a reinforced concrete column. After a plastic hinge is formed, the load path will change until the second plastic hinge is formed. The philosophy of ductility and the concept of plastic hinging are applicable to precast bridges if connections are monolithic.

WSDOT seismic design is based on the AASHTO Guide Specification for LRFD Seismic Bridge Design and modifications per BDM[3]. The displacement-based design is intended to achieve a "No Collapse" condition for bridges using one level of Seismic Safety Evaluation.

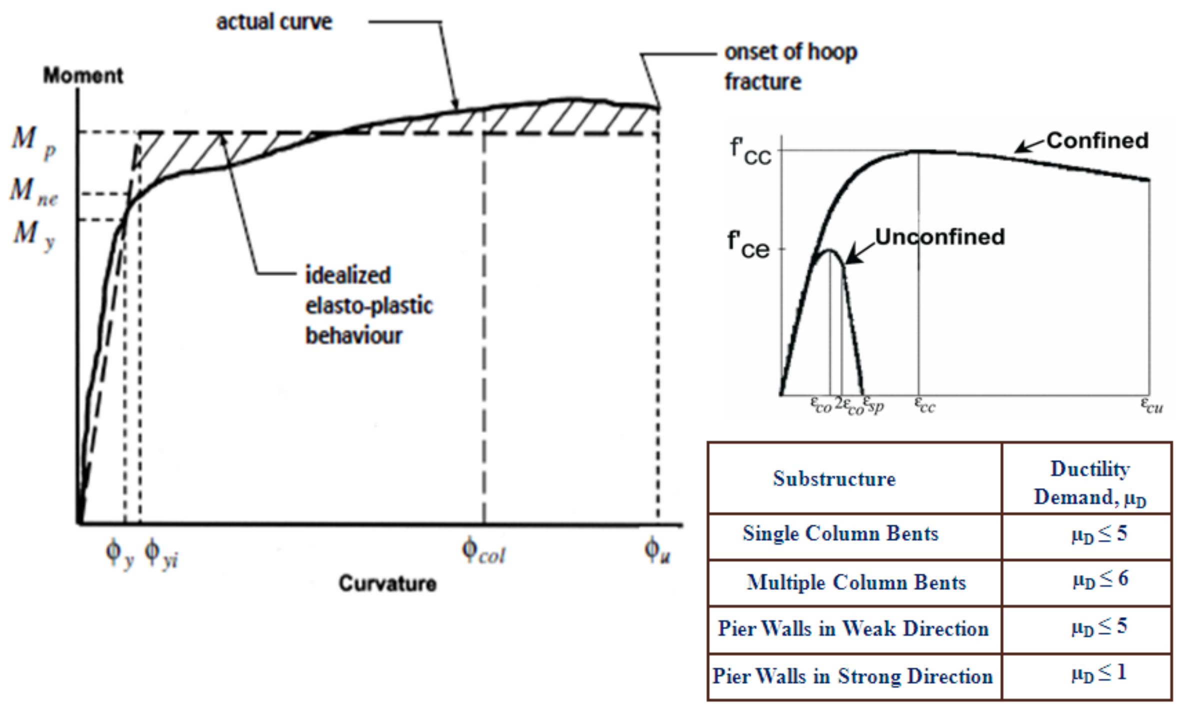
The displacement-based analysis is an inelastic static analysis using expected material properties of modeled members. Inelastic static analysis, commonly referred to as "push over" analysis, is used to determine the reliable displacement capacities of a structure or frame as it reaches its limit of structural stability.

The unconfined concrete compressive strain at the maximum compressive stress, ε_{co}, shall be considered equal to 0.002. The ultimate unconfined compression strain based on spalling, ε_{sp}, shall be considered equal to 0.005. The confined compressive strain, ε_{cc}, and the ultimate compressive strain, ε_{cu}, for confined concrete should be computed using Mander's model.

The plastic moment capacity of all ductile concrete members should be calculated by moment-curvature (M-φ) analysis on the basis of the expected material properties.

The moment-curvature analysis shall include the axial forces due to dead load, together with the axial forces due to overturning. The M-φ curve should be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross-section. The elastic portion of the idealized curve shall pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity should be obtained by equating the areas between the actual and the idealized M-φ curves beyond the first reinforcing bar yield point, as shown in Figure 2.

Figure 2 shows the stress–strain model for confined and unconfined concrete using Mander's model, idealized moment-curvature diagram, and ductility demand for concrete substructure components[2].



Substructure	Ductility Demand, μ_D
Single Column Bents	$\mu_D \le 5$
Multiple Column Bents	$\mu_D \le 6$
Pier Walls in Weak Direction	$\mu_D \le 5$
Pier Walls in Strong Direction	$\mu_D \le 1$

Figure 2: Stress–Strain Model, Moment-Curvature Diagram, and Ductility Demand

The reinforcing steel shall meet the specific properties provided in Table 1. The expected yield strength of 95 ksi is used for minimum development length requirement of grouted duct sleeves. WSDOT uses exclusively ASTM A706 reinforcing steel bars for all bridge applications.

Table 1: Stress Properties of Reinforcing Steel Bars

Property	Notation	Bar Size	ASTM A 706	ASTM A 615 Grade 60
Specified minimum yield stress (ksi)	f_y	#3– #18	60	60
Expected yield stress (ksi)	f_{ye}	#3– #18	68	68
Expected tensile strength (ksi)	f_{ue}	#3– #18	95	95
Expected yield strain	ε_{ye}	#3– #18	0.0023	0.0023
Onset of strain hardening	ε_{sh}	#3– #8	0.0150	0.0150
		#9	0.0125	0.0125
		#10 & #11	0.0115	0.0115
		#14	0.0075	0.0075
		#18	0.0050	0.0050
Reduced ultimate tensile strain	ε_{su}^{R}	#4– #10	0.090	0.060
		#11– #18	0.060	0.040
Ultimate tensile strain	ε_{su}	#4– #10	0.120	0.090
		#11– #18	0.090	0.060

The procedure outlined below is for displacement-based analysis and is applicable to bridges made of precast components. The basic assumption is that the displacement demand obtained from linear-elastic response spectrum analysis is an upper bound of the displacement demand, even if there is considerable nonlinear plastic hinging.

1. Perform linear elastic response spectrum analysis of the bridge based on design acceleration spectra specified by national or local specifications.
2. Determine the lateral and longitudinal displacement demands.
3. Calculate the moment-curvature diagram for each column and from that, the elastic and plastic and ultimate curvatures. Moment-curvature diagram of cracked concrete may be considered if cracking of precast girder to diaphragm connection is evident.
4. Using the above information and pier geometry (single or multi-column configuration), compute the displacement ductility of each column and ultimate displacement capacity.
5. Perform pushover analysis of each pier in transverse direction. Also, perform pushover analysis of the bridge in longitudinal direction. For this purpose, the plastic hinging moment for each column must be computed, and it might be necessary to incorporate foundation flexibility as well.
6. Compare the total displacement capacity of the pier to the displacement demand. If the capacity is insufficient, then higher ductility is required.
7. Design the superstructure and foundation for 20% higher capacity than the plastic capacity of the columns to make sure that plastic hinges occur within the column.

PRECAST BENT CAP SUITABLE FOR SEISMIC REGIONS

During the initial development of precast bent system, three main issues should be considered:

1) Constructability of the system,
2) Development of the large bars within the space available, and
3) Seismic response of the precast connection compared with that of a typical cast-in-place reinforced concrete system.

A precast bent system with grouted duct moment resisting connection was used for the conceptual design. Using precast elements with a small number of bars and ducts, it is possible to assemble a bridge bent quickly. The connection between column and cap beam is made with large bars that project from the top of the column and are grouted into ducts in the cap beam. The advantage of a small number of large bars is the reduction in the number of alignments needed. The proposed system uses large diameter ducts to maximize assembly tolerances.

During the initial development of the proposed precast system, the anchorage length required for large bars that could exceed the space available in typical cap beams was a concern. Development of these bars is particularly demanding under the cyclic loads caused by earthquakes. To address this concern, the University of Washington[5] (UW) has performed 14 pullout tests with bars as large as #18. The tests and accompanying nonlinear finite element analyses showed that large bars confined by ducts and typical cap beam reinforcement can develop their yield and fracture stresses in as little as six and ten bar diameters, respectively.

Full scale monotonic pull-out tests, with different embedment lengths, were first conducted to investigate the bond characteristics of large bars grouted into corrugated ducts. These tests confirmed that the #18 bars could be developed in the depth of the cap beam. The specific of the connection test, such as column hysteresis graphs and plastic hinging formation, are presented in the UW report.

The precast bent with the large-bar system should have the same seismic performance as a typical cast-in-place reinforced concrete system. The cyclic tests were performed at UW on three variations of the large bar precast, as well as a typical cast-in-place connection for comparison. All three variations of the proposed system performed satisfactorily to a drift ratio of 5.5 percent, before 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 system. This finding suggests that the large-bar, large-duct precast system has sufficient ductility capacity for all foreseeable seismic demands.

Details of the cap beam-column connection consist of six #18 vertical column steel bars grouted into 8 in. diameter corrugated metal ducts embedded in the cap beam, as shown in Figure 3. Precast concrete columns with six bars protruding are brought onto site, braced, and then cast integrally with their footing. Later, the precast cap beam is fitted over the column bars through the corrugated ducts and grouted in place, completing the bent substructure. The small number of bars and the generous tolerance in the connection lead to good constructability, but the structural integrity of the connection depends on the anchorage of the bars in the ducts. The purpose of the selection of six #18 vertical column bars was to reduce the congestion at column to cap connection while providing adequate tolerances for precast construction. Figure 3 shows the UW setups of large bar-duct pullout and column-to-cap-connection specimens. The grouted bar-duct concept was used for column-to-cap-connection.

Figure 3: UW Test setups of large bar-duct pullout and column-to-cap-connection

The grouted duct concept was applied to a three-span prestressed precast concrete bridge in high seismic zone of western Washington. The project increases mobility and safety within the growing metropolis. This project is the first application by the highway owner that uses precast concrete for bridge girder support precast bent caps. Based on the project success, the owner anticipates incorporating this method as an available practice for future designs. The bridge site is an extremely 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 intermediate pier bent caps in lieu of the cast-in-place requirements in the contract plans. This change would save the owner and the contractor several weeks on the contract duration. Figure 4 shows the placement of a precast bent cap using the grouted duct concept in a bridge in Washington.

Figure 4: Precast Bent Cap under Construction in Washington State

The bridge uses wide flange WF74G girders to span over a railroad right of way and an urban arterial. Precast concrete girders were the best choice for the superstructure. They are durable and have low maintenance and lifecycle costs. Precasting the girders increases the public's safety and convenience during construction by minimizing road closures and eliminating false-work over traveled lanes. The substructure cross beam was precast in order to save construction time. The use of precast concrete made duplicating the cast-in-place design feasible. As shown in Figure 4, the 14 #14 column bars went through the 4 in. (100 mm) galvanized steel ducts placed in the precast bent cap using a template.

IMPLEMENTATION OF RESEARCH OUTCOME

The UW pullout test results are shown in Figure 5. The characteristics of materials included ASTM A706 Grade 60 deformed reinforcing bars, corrugated galvanized standard post-tensioning ducts, and cementitious grout with compressive strength of 8.0 ksi (56 MPa). The corrugated pipes are available in diameters from 6 in. (152 mm) to 12 ft. (3.7 m). The helical corrugations of such pipes are deeper and the bond properties are potentially better than those of standard PT duct. Figure 5 shows the effect of the bar size on stress-displacement responses from tests with No. 10, 14, and 18 bars with the same normalized embedment length of eight bar diameters. To compare results from tests conducted with different bar sizes, the displacement was normalized by the bar diameter[5].

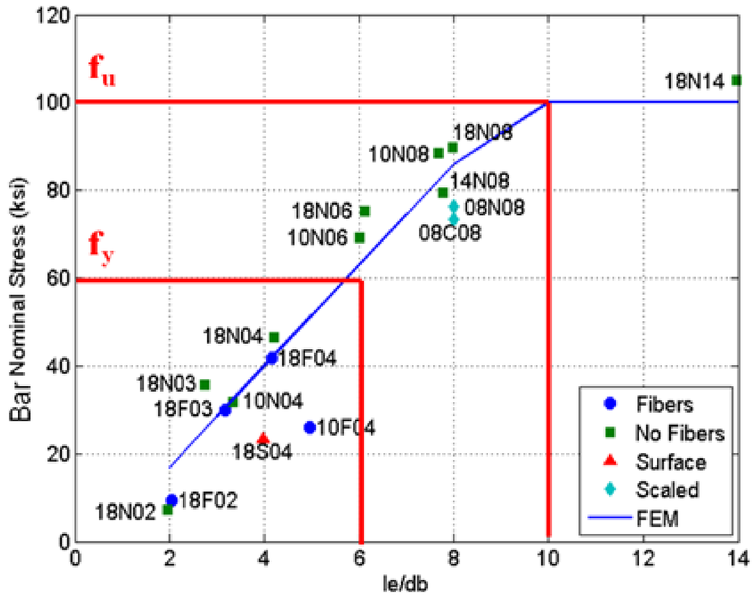


Figure 5: UW grouted Bar-Duct Pullout Test Results

(18N03 represents 3 tests of #18 bars in grouted ducts; 18F03 represents 3 tests of #18 bars in fiber grouted ducts)

The presence of fibers in the grout reduced the compressive strength of the grout for each pair of parallel tests conducted with and without fibers. Fibers also reduced the peak average bond stress for three of the four pairs. 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 on average.

The bar yielded in the tests with embedment lengths of $6d_b$ or more. Inelastic elongation is accompanied by a reduction in bar 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.

DERIVATION OF SEISMIC DEVELOPMENT LENGTH EQUATION

A design equation was developed based on the data shown in Figure 5. The development length equation shares the same dependencies on steel strength, bar diameter, and concrete or grout strength with the AASHTO development length equations. The proposed grouted bar-duct development length in ksi units is shown in Equation 1.

$$L_d = 1.5\left[\frac{f_s}{4\sqrt{f'_g}}d_b + \frac{d_{duct} - d_{bar}}{2}\right] \qquad (1)$$

The second term represents the effect of the cone and the difference between duct and bar diameters. Equation 1 is conservative compared with the test results, but nonetheless results in much shorter embedment lengths for a given bar stress than those provided by the AASHTO LRFD Specifications Article 5.111.2.1 for deformed bars in tension, as shown in Table 2. The expected tensile strength tensile strength is 95 ksi (665 MPa), and compressive strength of grout is 8.0 ksi (42 MPa).

Table 2: Proposed Grouted Bar-Duct Development Length

Bar Size	Bar 0utside diameter d_{bar}, in.	Duct Diameter d_{duct} In.	Grouted Sleeve Development Length, L_d In.	L_d/d_b Sleeve	AASHTO Development Length In.
3	0.42	2	6.47	15.42	15.96
4	0.56	2	8.13	14.52	21.28
5	0.70	3	10.54	15.06	26.6
6	0.83	3	12.08	14.56	31.54
7	0.96	4	14.37	14.97	36.48
8	1.10	4	16.03	14.57	46.91
9	1.24	6	19.19	15.47	59.38
10	1.40	6	21.08	15.06	75.41
11	1.55	8	24.36	15.72	92.63
14	1.86	8	28.03	15.07	166.25
18	2.48	8	35.38	14.26	166.25

Seismic modification factors to Equation 1 are needed to account for the reduction in bond strength due to cycling. AASHTO LRFD requires increases in development length of 25 percent. For practice, a seismic modification factor of 1.5 would be conservative and would result in a seismic development length of 10 bar diameters or longer.

Table 3 shows the WSDOT adoption of minimum embedment length requirement for grouted bar-duct sleeves. The minimum development lengths are based on ASTM A706 Grade 60 deformed reinforcing bars with expected tensile strength of 95 ksi (665 MPa), and compressive strength of 6.0 ksi (42 MPa) for all practical reasons.

Table 3: WSDOT Minimum Embedment Length Requirement

Bar Size	Duct Diameter d_{duct} In.	Grouted Bar-Duct Development Length, L_d In.
3	2	8.0
4	2	10.0
5	2	12.0
6	3	14.0
7	3	16.0
8	3	18.0
9	4	21.0
10	4	24.0
11	4	27.0
14	6	32.0
18	6	40.0

WSDOT HFL PROJECT

The WSDOT HFL project demonstrates that bridges can be built economically, rapidly and safely in seismic regions. As part of the HFL project[6], the UW has performed several tests of precast column-to-footing connection. To achieve proper interface shear transfer between the precast column and the cast-in-place concrete footing, the exterior of the column is roughened near the bottom to improve the transfer of shear stress. The shape of the lower column segment extending into the footing is changed from circular to octagonal to provide more uniform interface surface. The precast column extends just below the footing, to assure that the force transfer at the bottom of the column bars can take place satisfactorily. Figure 6 shows the UW test setups for precast column-to-footing connection. The connections described in Figure 6 are designed to be used with a precast bent system that includes precast columns and a precast cap beam.

Figure 6: UW Test setups for precast column-to-footing connection

Following the UW 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, I-5. The demonstration project is a replacement bridge that will be built on an alignment parallel to an existing bridge. It is a two-span bridge with tall abutments on the end and a center bent that is located in the median strip. The details of WSDOT HFL project is shown in Figures 7 through 9. The bridge features are listed below:

- Unique connection to footing
- Precast columns in segments
- Column segment grouted joints
- Precast bent cap segments
- Cast-in-place precast bent cap closure
- Precast superstructure with CIP closure at intermediate pier
- Precast end and intermediate diaphragms

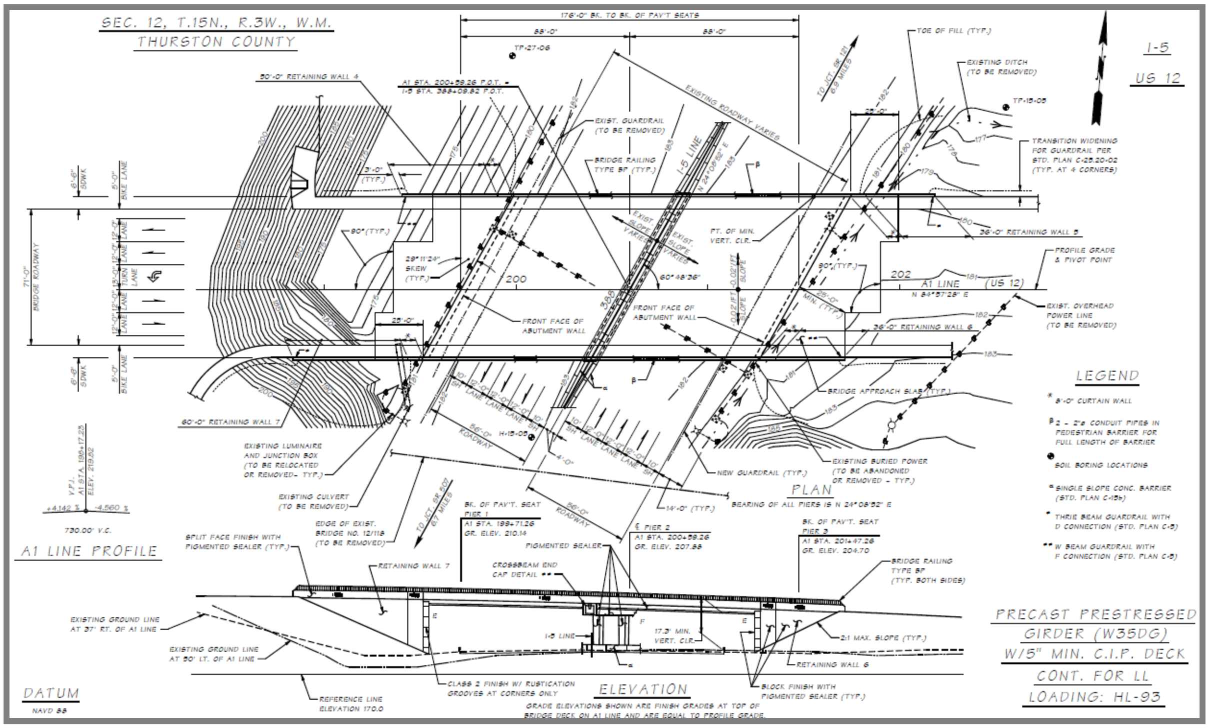


Figure 7: HFL Bridge Layout

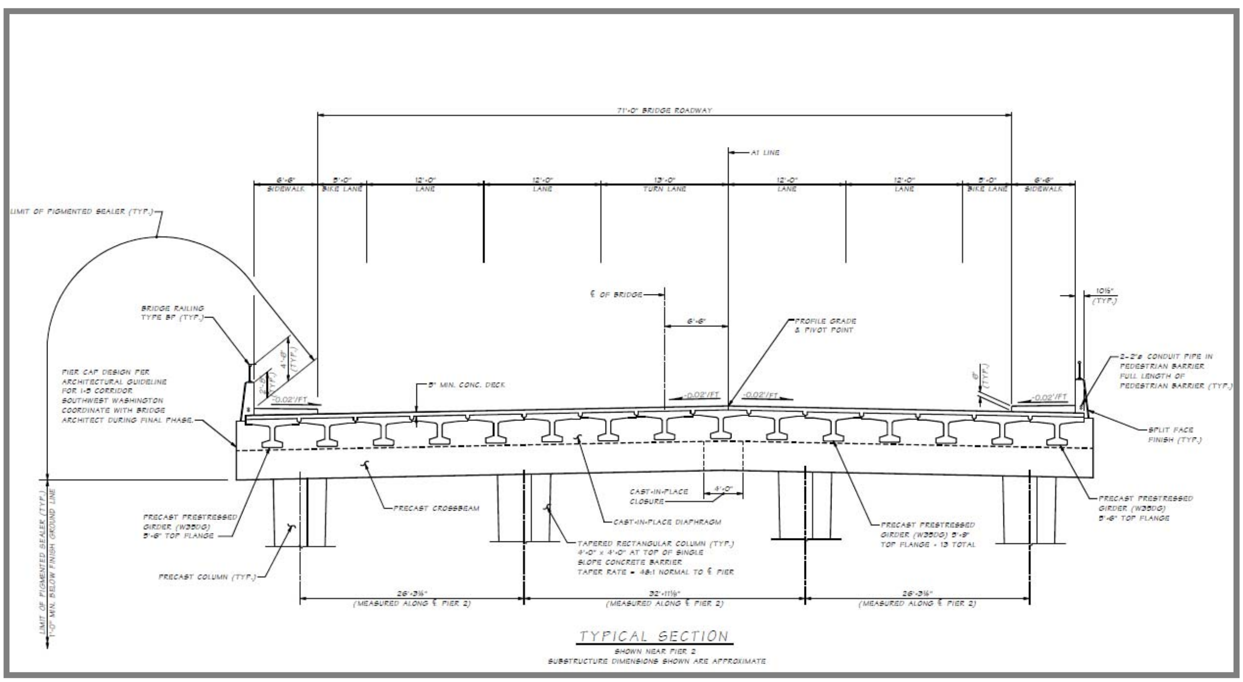


Figure 8: HFL Bridge Typical Section

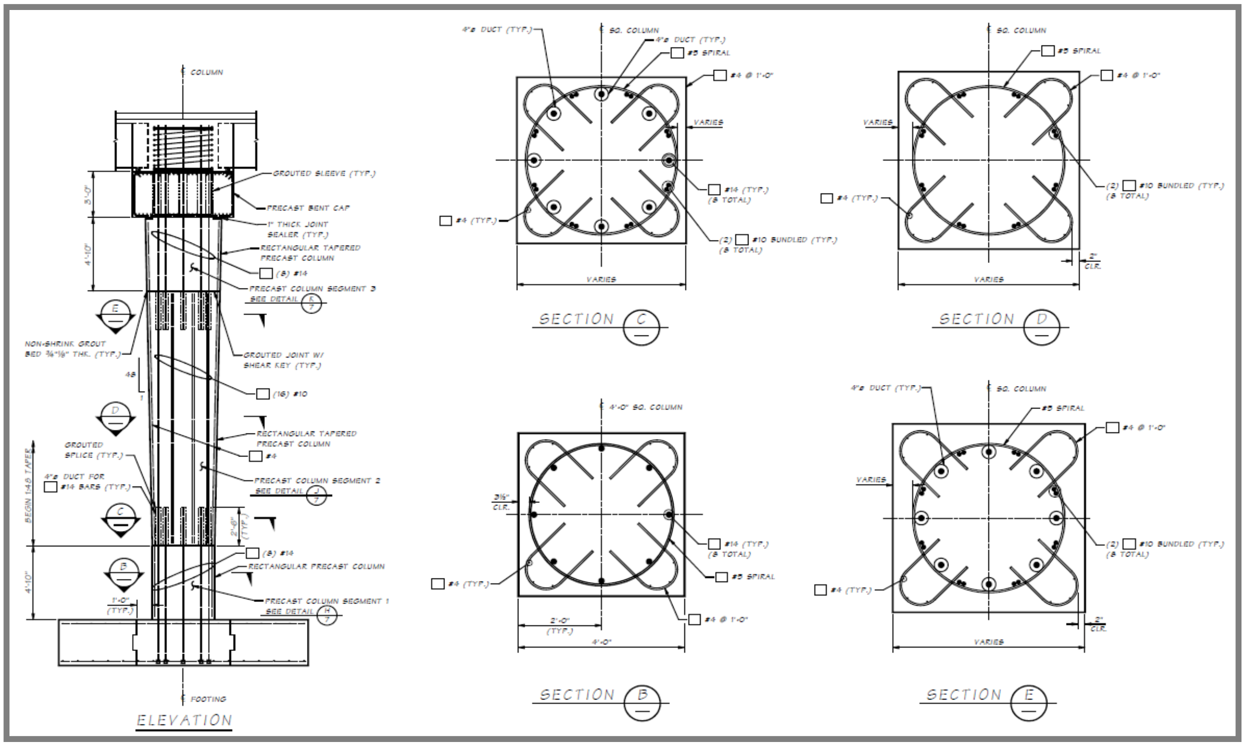


Figure 9: HFL Bridge Segmental Column Details

The top of footing reinforcement is not continuous throughout the precast column segment, as it is usually done with the cast-in-place applications. To achieve proper interface shear transfer between the precast column and the cast-in-place concrete footing, the exterior of the column is roughened near the bottom to improve the transfer of shear stress. The shape of the lower column segment extending into the footing is changed from square to octagonal to provide more uniform interface surface.

The construction sequences for the placement of precast column segment into the cast-in-place footing are shown in Figure 10 and listed below:

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

Figure 10: Construction sequences for placement of precast column segment into footing

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.

The construction sequences for placement of precast column segments and bent cap are shown in Figures 11 and 12 and listed below:

1. Place and shim middle column segments
2. Place and shim top column segments
3. Install column bracing
4. Place and shim precast bent cap segments

Figure 11: Column Segment Placement

Figure 12: Precast Bent Cap Placement

The precast bent system to be used in the HFL 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, 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 pick and shipping weight restrictions led to the two-piece solution. This decision could, of course, vary by project.

Grouting the joints between column segments and the column to bent cap is shown in Figure 13 and 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

Figure 13: Grouting the Joints between Column Segments and the Column to Bent Cap

The superstructure of the bridge is comprised of 35 inch deep decked-bulb tee prestressed girders that span 88 feet. These are supported by the center bent comprised of spread footings, precast column segments, a precast dropped cap beam and a cast-in-place diaphragm with a 5 inch cast-in-place topping over the decked bulb tees, whose flanges act as stay-in-place forms.

The construction sequences for placement of precast superstructure are shown in Figure 14 and listed below:

Superstructure Placement:

1. Place precast girders on oak blocks
2. Install girder bracing
3. Complete welded ties between girders
4. Place slab reinforcement and cast concrete
5. Cast pier diaphragm concrete ten days after slab casting
6. Cast traffic barrier and sidewalk

Figure 14: Placement of Precast Girders and Casting Slab Concrete

LESSONS LEARNED FROM THE HFL PROJECT

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

1. Tolerance of precast pieces was not consistent with survey tolerances
2. Pressure from grouting may lift segments
3. Shim locations and grout lifting pressures need to be included in erection plan calculations
4. Grout form quality and ability to seal with column is key to successful grouting
5. It would be helpful during inspection and grouting if grout tubes were built/mapped as part of the precast operation.
6. Transverse bars in the closure should be made from single length bars with 135 degree bends and 90 degree bends instead of “U” shaped stirrups.

7. The contractor indicated that they would have preferred the columns to be cast in place; however, they could see the benefit to using a single precast column with the grout connection at the precast bent cap only. This would eliminate the cure time for the concrete and require bracing for only one day. With the installation of all the segments and precast bent cap prior to grouting they needed to provide bracing for an extended amount of time.
8. The contractor preferred the joints that had the ducts in the lower section. They indicated that all the joints where the ducts were below the joint were grouted without any leaking. They believe that this configuration made it so there was a much smaller amount of pressure on the grout forms since the forms did not have to resist the pressure head from the ducts. The superintendent suggested that the joint be sealed by having something such as a compression seal sandwiched between the top and bottom segments, which would be held in place by the weight of the segment on top.
9. The contractor felt that the grouted joint between the column and the bent cap was the easiest to construct, although they would have preferred to have this joint larger.
10. The contractor’s concern here was that the lap splice between the two precast bent cap segments would hit and they would not be able to properly align the pieces. They did not have a specific reason other than to point out all the tolerances that could add to alignment error (column tolerances, bar placement in the columns, orientation of the columns, cross-slope of the cap, placement of the ducts in the cap, slope of the ducts in the cap, etc.).
11. It was apparent during the installation of the stirrups in the cross beam closure that the stirrups should have been detailed as ties. It was nearly impossible to install the stirrups as shown in the plans, and a modification to the stirrups was made accordingly. A three legged stirrup should be detailed as three ties to facilitate placement. The contractor indicated that the closure was very congested and would have preferred to use a pea gravel concrete mix design to make it easier to place and consolidate. They thought the hanging form work was simple and easy to construct.
12. The contractor felt that the 5 in. precast walls at pier 2 cross beam were not particularly useful. They indicated that they still need to install formwork at the bottom of these walls to the top of the precast bent cap and around the oak blocks. They felt that this diaphragm pour would be much simpler if it were cast in place and that it would take the same amount of time with or without these walls. They also communicated that the precast end panels were not very helpful in speeding up the work, and would prefer this to be cast in place since they had to make forms regardless.

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 use of precast bent caps results in cost savings by eliminating the need for elevated false work and its foundation. It also improves workers’ safety, as rebars and concrete

can be placed at the ground level.

2. The column-to-cap beam connection is made with a small number of large 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. Precast prestressed concrete bridge systems are economical and effective for rapid bridge construction. Precasting eliminates traffic disruptions during bridge construction while maintaining quality and long-term performance.
4. The development length of a reinforcing bar grouted into a corrugated steel pipe is much shorter than suggested by current code equations. A simple equation based on this research results in development lengths of typical grouted bar-duct sleeve connections.

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