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Replacing the Boeing North Bridge

- The Boeing North Bridge is a multispan girder bridge used to transport completed aircraft to Renton Municipal Airport in Renton, Wash., where they undergo final inspection before takeoff.
- To meet the constraints imposed by Boeing's production schedule, environmental regulations, winter construction, airport operations, and the surrounding community, the new bridge was designed using precast concrete columns, partially precast concrete crossbeams, and full-depth precast concrete deck panels.
- Due to the high seismicity of the site, the connections between the substructure elements had to be seismic resisting.

Precast concrete facilitates accelerated bridge construction by allowing bridge components to be constructed concurrently or ahead of on-site work activities and by reducing on-site construction time. For the Boeing North Bridge in Renton, Wash., accelerated bridge construction was of interest because of a planned increase in Boeing's production.

The original bridge and approach aprons, built in 1969 and 1940, respectively, had been determined to be seismically deficient. The Boeing North Bridge was critical to Boeing's Renton factory because it provided a means to transport aircraft across the Cedar River to the Renton Municipal Airport. Without access to the airport, the planes cannot leave the factory. Boeing wanted the bridge replaced before it increased production.

Numerous constraints affected the construction schedule.

- **Uninterrupted production.** On average, two aircraft cross the bridge each day at times that are difficult to predict, necessitating demobilization of construction activities and equipment.
- **Airport operations.** The project is located within the object-free area of the Renton Municipal Airport. The higher an object projects above the bridge deck, the shorter the effective runway length. Close coordination with the airport minimized effects on their operations. However, airport operations imposed significant restrictions on the contractor's schedule, particularly for pile driving, pile extraction, and shaft construction.
- **Environmental.** The Cedar River is a salmon habitat and is considered environmentally sensitive. The in-water work window (also called the fish window) was limited to the 2½ months between June 1 and August 15.
- **Noise restrictions.** The contract imposed a number of noise restrictions and mitigation strategies. Noise mitiga-

The Boeing North Bridge project site is situated between Boeing's Renton, Wash., factory and the Renton Municipal Airport. Aircraft cross the bridge from the factory for final inspection before takeoff. Courtesy of Greg Banks..

tion costs were accrued due to the need to perform work at night based on the airport's operations.

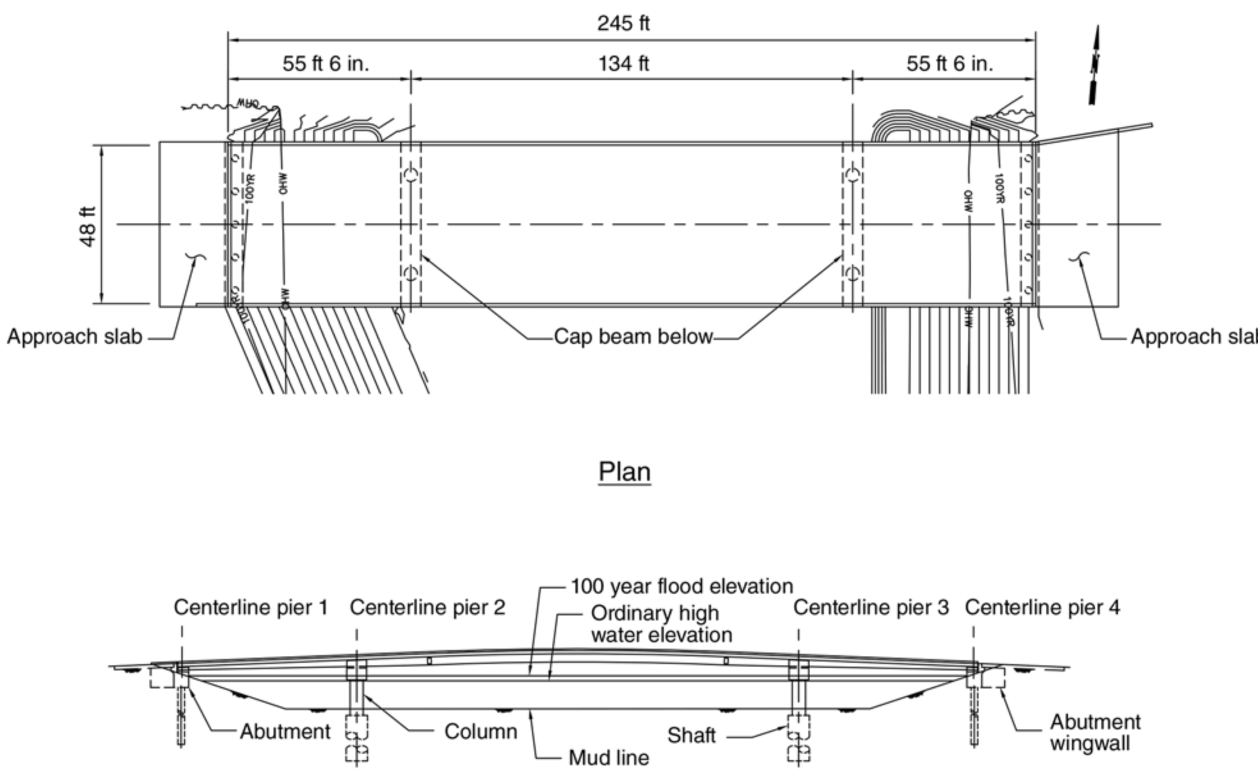
The bridge had to be completed by January 2015. Work began in May 2013 and spanned three in-water work windows.

Overall bridge description

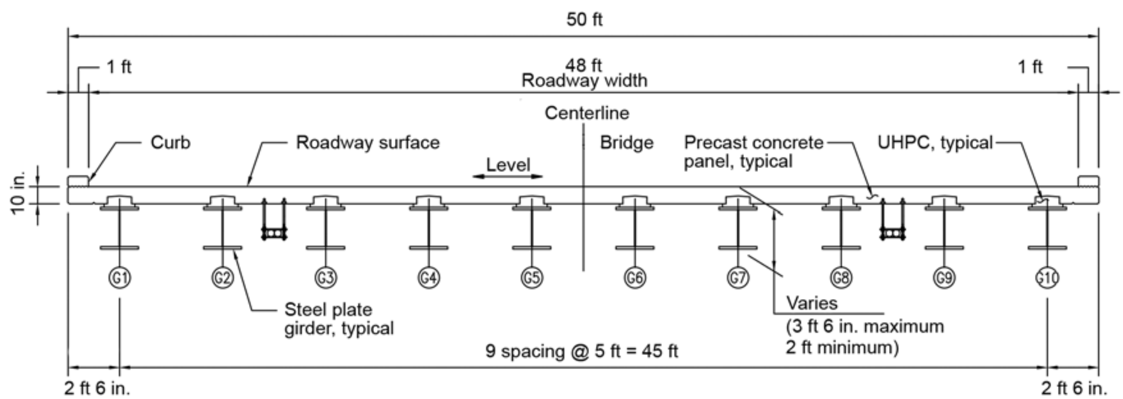
The bridge is a three-span continuous structure that is 245 ft (74.7 m) in total length. Each end span is 55.5 ft (16.9 m) long, and the main span is 134 ft (40.8 m) long. The bridge superstructure comprises 10 variable-depth steel plate girders that span continuously between end piers. Each girder is pinned with disk bearings to the dropped crossbeam at the intermediate pier locations. The bridge has a 48 ft (14.6 m) wide travel way and measures 50 ft (15.2 m) in total width. Full-depth precast concrete deck panels, made composite with the steel plate girders, span between the girders.

Design criteria

The bridge was designed to comply with the American Association of State Highway and Transportation Officials' (AASHTO's) *LRFD Bridge Design Specifications*[1] and the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*.[2] Aircraft live loadings were considered in addition to the HL-93 live loading, though they were not considered to act concurrently. The aircraft loading was assumed to be the operating empty weight of the aircraft. Horizontal impact on the curb and railing was not considered, nor were aircraft braking forces. The dynamic allowance for vertical impact for aircraft



The profile of the bridge was governed by aircraft towing grade restrictions and the required clearance over the 100-year flood elevation. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m. Courtesy of Greg Banks..



Typical section for the new Boeing North Bridge. The variable depth of the structure satisfied clearance requirements over the 100-year flood elevation and met aircraft towing grade restrictions. Note: UHPC = ultra high performance concrete. 1 in. = 25.4 mm; 1 ft = 0.3048 m. Courtesy of Greg Banks.

loading was taken as 33%. The load magnification γ for the aircraft loading was taken as 1.35. Ninety-five percent of the operating empty weight acts on the rear wheels and is equally distributed between the rear wheel struts. It was assumed that all aircraft are towed across the bridge by a tug positioned 21.4 ft (6.53 m) in front of the aircraft front wheels.

Intermediate pier configuration

The intermediate piers consist of cast-in-place concrete drilled shafts, precast concrete columns, and a two-stage dropped crossbeam.

Crossbeams

Each crossbeam was formed by casting concrete into a tub-shaped precast concrete form. The crossbeams were 50 ft (15.2 m) long, 6 ft (1.8 m) wide, and 3.5 ft (1.1 m) deep. The precast concrete tub weighed 85,000 lb (39,000 kg), within the capacity of readily available cranes.

The overall depth of the crossbeam was 3.5 ft (1.1 m), and the spacing between columns was 30 ft (9.1 m). The shallow depth of the crossbeam was governed by grade restrictions imposed by Boeing and the water surface elevation of the Cedar River. Using the 4% maximum allowable bridge grade and a 3.5 ft crossbeam depth, the soffit of the crossbeam was 0.1 ft (30 mm) above the ordinary high-water elevation.

The crossbeam was 2 ft (60 mm) wider than the column diameter to satisfy the prescriptive requirements of AASHTO.[2]

The precast concrete portion of the crossbeam was both pretensioned and posttensioned. The pretensioning consisted of six 0.6 in. (15 mm) strands and was required to carry the self-weight of the precast concrete portion and the weight of the fresh cast-in-place concrete infill. Tensile stresses of $3\sqrt{f_c'}$ psi ($0.25\sqrt{f_c'}$ MPa) were temporarily allowed in the precast concrete portion for placement of the cast-in-place infill. The posttensioning consisted of four tendons comprising twelve 0.6 in. (15 mm) strands in the precast concrete portion. The tendons were profiled to efficiently resist the moments due to permanent superstructure loads, live loads, and seismic demands. The thickness of the precast concrete walls was increased locally in the crossbeam cantilevers from 1 ft (300 mm) to 1 ft 10 in. (560 mm) to accommodate the posttensioning anchors and the lateral sweep of the tendons. The composite crossbeam was designed to remain entirely in compression in service. Shear interface steel was provided between the cast-in-place concrete infill and the precast concrete portion to resist the full flexurally-induced tensile stresses.

A grouted duct connection was used to connect the precast concrete column to the crossbeam. The precast concrete portion of the crossbeam included corrugated steel column bar ducts with confinement steel in a 5 ft (1.5 m) wide solid section centered on each column. The solid section terminated 9 in. (230 mm) from the top of the crossbeam to facilitate placement of bar heads on the ends of the protruding column bars and of the longitudinal top mat reinforcing steel.

Bearing anchorage assemblies under each girder at the intermediate piers anchor into the dropped crossbeam. They were located primarily in the cast-in-place infill portion of the crossbeam to minimize concerns regarding placement tolerances.

Columns

There are two intermediate piers, each with two columns. Each column has a diameter of 4 ft (1.2 m) above the top of the shaft. The total column length was 19.59 ft (5.97 m). The lower 9 ft (2.7 m) of the column was cast into the 6.5 ft (2.0 m) cast-in-place concrete drilled shafts to form the column-to-shaft splice. Within the splice zone, the column section changed from circular to octagonal. Each side of the octagonal section was formed with a 1 in. (25 mm) sawtooth pattern along the full height of the column-to-shaft splice zone. The vertical column reinforcement consisted of eight no. 14 (43M) bars. The

The seismic-resistant intermediate piers comprise precast concrete columns and partially precast concrete crossbeams. Connections developed as part of the Highways for Life program were used to connect the columns and crossbeams. The crossbeams were partically precast to limit their pick weight. Courtesy of Greg Banks.

horizontal reinforcing steel consisted of a no. 6 (19M) spiral at a 4 in. (100 mm) pitch. The vertical bars projected 3.17 ft (0.97 m) from the top of the column to facilitate the connection with the crossbeam.

Seismic-resisting bent system

The seismic design of the bridge was conducted using a displacement-based approach in accordance with the AASHTO *Guide Specifications for LRFD Seismic Bridge Design*. The bridge was designed as a Type 1 earthquake-resisting system, or a ductile substructure—the columns—with an essentially elastic superstructure. In this system, transverse seismic forces generate high moments and shears as well as large inelastic cyclic strain reversals in the ductile elements. The peak forces occur at the top and bottom of the column and at the bottom of the column when the pier is excited in a direction parallel or perpendicular to its centerline, respectively. With the plastic hinge locations corresponding with the connections at the tops and bottoms of the columns, the seismic resistance of the bent system depends heavily on the connection details between the ductile columns and the capacity-protected crossbeam and drilled shaft. The connections needed to be robust to accommodate the inelastic cyclic deformations while also facilitating construction.

The column-to-crossbeam and column-to-shaft connections were defined as grouted duct connections and socket connections, respectively. The connection terminology and design basis were taken from work conducted as part of the Highways for Life program, which developed design and construction guidelines based on experimental testing and field implementation.[3,4]

Grouted duct connection

To connect the column to the crossbeam, a vertical column bar was grouted into an oversized duct in the precast concrete portion of the crossbeam. Force is transferred from the vertical column bar to the grout via the duct to the surrounding concrete. Semirigid corrugated steel ducts with a 4 in. (100 mm) inner diameter were cast into the precast concrete portion of the crossbeam. The diameter of the duct was chosen to provide adequate construction tolerance and was within the recommended duct-to-bar diameter ratio of 6 or less specified by the Highways for Life program. Eight no. 14 (43M) bars were used in the columns to minimize the number of bars that needed to be anchored in the ducts. The vertical column bars extended beyond the ducts in the precast concrete portion of the crossbeam, terminating directly below the top mat of reinforcing steel in the cast-in-place concrete portion. After placement of the precast concrete portion of the crossbeam, heads were threaded onto the ends of the vertical column bars, which were anchored as high as possible in the crossbeam to maximize the resistance of the joint to shear forces and transferring forces between the vertical column bars and the horizontal crossbeam bars.

Socket connection

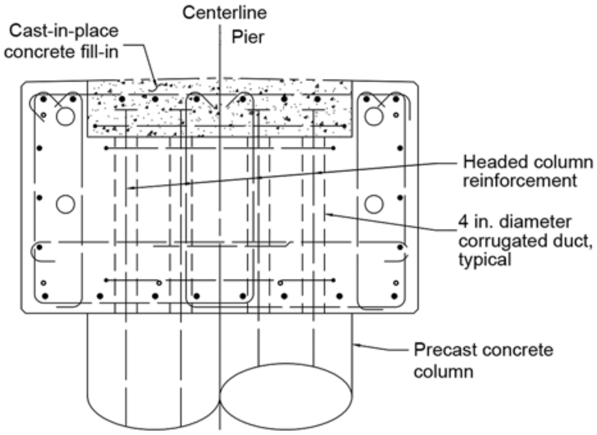
A socket connection between the column and the drilled shaft was formed by embedding the column into the drilled shaft before casting the upper portion of the drilled shaft. Force transfer occurs through shear across the surface of the socket and by prying action. The cross section of the embedded portion of the column was octagonal, in accordance with the Highways for Life program, to maximize the area for interface shear transfer. The column surface was sawtoothed with 1 in. (25 mm) teeth. The initial concrete placement in the drilled shaft was terminated 3 in. (75 mm) below the eventual elevation of the bottom of the column. A grout pad filled the gap, providing a direct bearing surface.

Because all of the vertical column bars were spliced at the same elevation, AASHTO Class C splice requirements were followed to determine the splice length, which was governed by the size of the reinforcing steel in the drilled shaft. Additional length was added to account for the noncontact lap splice distance between the column and shaft bars.

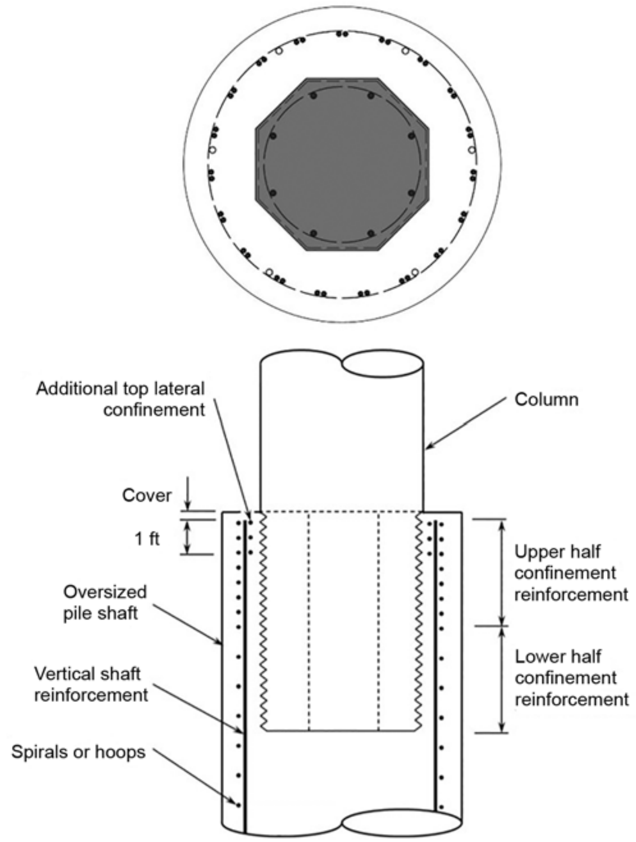
Lateral confinement of the column-to-shaft connection was provided by spiral reinforcing steel and a permanent casing. Additional confinement was required over the upper portion of the splice region to resist tension due to prying action of the column in flexure. The permanent casing extended over the length of the splice. The splice was located below the mudline; the casing was used to hold back the soil. As a result, it was decided to use the permanent casing in design. To ensure that the permanent casing was engaged, a split shear ring was added near the top. The thickness of the casing was increased to ¾ in. (19 mm), which was sufficient for the confinement demand to be carried by the permanent casing alone and provided a ⅛ in. (3 mm) corrosion allowance.

Full-depth precast concrete deck panels

The full-depth precast concrete deck panels were 50 ft (15.2 m) long (parallel to the transverse axis of the bridge), 8 ft (2.4 m) wide, and 10 in. (250 mm) thick. Of the 30 panels, 28 were identical and the two end panels were unique.



This grouted duct connection was used to connect the column to the crossbeam. Large-diameter reinforcing steel projects above the top of the column through oversized corrugated steel ducts embedded in the precast concrete crossbeam. Use of large-diameter bars minimizes the number of ducts required. Courtesy of Greg Banks.



The column-to-shaft connection was formed by use of a seismic-resistant socket connection. The octagonal precast concrete column was set into the drilled shaft prior to casting the upper portion of the drilled shaft. Socket connections require additional reinforcing steel in the top of the shaft to resist tension due to prying action. Courtesy of Lee Marsh.

The panels were designed as one-way slabs spanning transversely between girder centerlines. Concentric pretensioning using twelve 0.6 in. (15 mm) strands in the panel transverse direction maintains each panel in compression under service conditions.

The panels were also posttensioned using a total of 20 continuous 4-strand tendons from end to end of the bridge to maintain compression across the transverse joints under service conditions.

The transverse panel joint was diamond shaped and continuous from end to end of the panel. Rectangular blockouts at the ends of each posttensioning duct facilitated splicing. The transverse panel joints were filled with a nonshrink grout.

Over each steel plate girder, a hidden pocket in the panel served as the joint between panel and girder. Transverse bottom mat reinforcing steel from the panel projected through the pocket, as did the girder shear studs, to transfer shear across the flange and panel interface. To increase the interface shear resistance the pocket was filled with ultra-high-performance concrete through a 4 in. (100 mm) hole after posttensioning. This was done to maximize the effectiveness of the prestressing across the transverse panel joints and to avoid introducing undesirable stresses in the steel plate girder.

Each panel has 20 leveling bolts to adjust the elevation of the top of the deck panel. The leveling bolts also aid in distributing the panel weight to each girder line.

Benefits of using precast concrete

For Boeing, schedule was paramount. Precast concrete deck panels, columns, and crossbeams lent flexibility to a highly constrained schedule. The use of precast concrete columns and crossbeams minimized in-water work time and made it possible to complete the work within the fish window. It was estimated that the use of precast concrete columns and crossbeams saved 10 to 20 workdays compared with conventional cast-in-place concrete construction methods.

The construction schedule for a conventional cast-in-place concrete deck depends on weather. The bridge deck was scheduled to be cast in late fall and early winter, which is western Washington's wettest season. It was estimated that the full-depth precast concrete deck panel system could be constructed within two to three construction weeks, including time required for posttensioning, grouting, and closure placements. The equivalent cast-in-place concrete bridge deck was estimated to take approximately two months in the anticipated weather conditions.

The Boeing North Bridge has a low profile and spans environmentally sensitive waters. The use of prefabricated substructure elements minimized the amount of concrete being cast over water whereby formwork would have been at or near the water surface.

Site implementation of precast concrete bridge elements

The columns and crossbeams at the intermediate piers were within the wetted perimeter of the Cedar River and thus were subject to restrictions of in-water work from June 1, 2014, to August 15, 2014. Work to be completed in this period included demolition of the existing bridge foundations, construction of temporary work platforms, installing the drilled shafts, erecting columns and crossbeams, removing temporary work platforms, and regrading the channel.

For the drilled shafts, temporary casings extended above the ordinary high-water elevation to keep river water out. Once the shafts were constructed up to the column connection, the columns could be erected. This entailed the following:

1. placing the concrete to approximately 3 in. (75 mm) below the bottom of the column elevation
2. cleaning loose debris from the joint
3. forming and placing the grout pad
4. attaching steel clips to the grout pad to guide setting of the column
5. setting the column and bracing it
6. aligning the reinforcing steel projecting from the top of the column with the duct openings in the crossbeam
7. casting the column-to-shaft splice removing the bracing and temporary casing

The construction sequence for placing of the precast concrete portion of the crossbeam on the columns required the following:

1. placing the form for the crossbeam soffit recess
2. placing the shim pack between column and crossbeam
3. erecting the crossbeam
4. grouting the ducts and the column-to-crossbeam interface

The grout for the ducts was poured from the top until it came up the ducts on the other side. Each duct was topped off. Air releases in the soffit form were plugged when grout was observed flowing out.

Once the grout reached its required strength, the remainder of the crossbeam could be constructed. Heads were threaded onto the ends of the projecting column bars. Reinforcing bars were placed in the infill section. Bearing anchorage assemblies were set, and the concrete was cast. Once the concrete gained the required strength, the whole section was posttensioned.

Construction of the superstructure began after the end of the second fish window. The steel plate girders were erected, followed by the full-depth precast concrete deck panels. The reinforcing bars and duct couplers were installed. Alignment, position, and cross slope were verified. The change in elevation at each girder centerline was measured to ensure appropriate load distribution to each girder. Last, the panels were adjusted to their final elevation.

The panels were erected beginning with the three panels at each end of the bridge. The end panels served as counterweights to keep the girders on their bearings while the other panels were erected. The remaining panels were placed from the middle outwards. Grout stops were placed at the outside edge of panels 15 and 16 over the centerline of each girder. Ultra-high-performance concrete was placed in the hidden pockets of panels 15 and 16. Then the transverse joints between all panels were grouted and the panels were posttensioned. Ultra-high-performance concrete was placed in the hidden pockets in the remainder of the deck panels beginning at the ends of the bridge and terminating adjacent to panels 15 and 16, respectively.

Concrete closure pours were completed at each end of the bridge. Then the bridge deck was ground to remove relief greater than ¼ in. (6 mm). Concrete curbs were constructed. Hold-down assemblies were installed at each end of the bridge. Then the bridge deck was overlaid with high-molecular-weight methacrylate.

Overcoming objections

Before this project, the general contractor was not familiar with the precast concrete seismic-resisting bent system. He submitted a change proposal to use cast-in-place concrete, citing concern that positioning the precast concrete crossbeam and aligning the column reinforcing bars with the ducts in the crossbeam would require personnel to work in close proximity to the overhead load, a potentially unsafe condition. However, the weight of the precast concrete portion of the crossbeam was approximately 85,000 lb (39,000 kg), comparable to that of a typical bridge girder. It was suggested that a good work plan be developed and that work platforms be built, or manlifts used, so that field personnel could be located to the side of the overhead work rather than directly underneath.

The intermediate pier column is about to be lifted and set into position. The corrugated portion of the column was embedded into the cast-in-place drilled shaft. Courtesy of Greg Banks.

The precast concrete elements required numerous grouted ducts and joints, most of which would have been unnecessary in cast-in-place concrete construction. The joint at the base of the column could have been replaced with a single construction joint at the top of the shaft. Grouting of the column-to-crossbeam interface and the vertical column reinforcing steel in the corrugated ducts in the column-to-crossbeam connection would have been obviated by cast-in-place concrete construction. The two-part crossbeam could have been made using one monolithic concrete placement. However, the precast concrete components were designed using cover requirements consistent with cast-in-place concrete. The construction joint at the top of the shaft proposed by the contractor could have been more susceptible to corrosion than the sawtooth construction joint. To minimize problems in grouting, a mock-up of this procedure was recommended.

Setting the precast concrete portion of the crossbeam so that the reinforcing bars projecting from the top of the column fit into the ducts in the crossbeam allowed little room for error. However, the drilled shafts could be out of position by as much as 8 in. (200 mm) without adversely affecting the precise positioning of the columns using survey equipment and templates. This is no different from cast-in-place concrete construction. The longitudinal reinforcement could be match cast with the ducts in the crossbeam at the precast concrete plant. The alignment of the no. 14 bars (43M) in 4 in. (100 mm) ducts allowed for tolerances typical of standard practice.

The precast concrete portion of the crossbeam is set in place on the precast concrete columns. The intermediate piers were constructed during the second fish window (summer 2014) within the wetted perimeter of the Cedar River. The use of precast concrete accelerated construction, allowing it to be completed during the allotted work window. Courtesy of Greg Banks.

The contractor submitted comparative schedules based on the use of precast concrete bridge elements and cast-in-place concrete construction. The two schedules showed the same construction duration of 38 days. The major difference between the two methods (as noted by the contractor) was that using precast concrete required a crane with a 120 ft (37 m) boom, while cast-in-place concrete did not. In addition to the added equipment costs, the boom height would adversely affect airport operations, so it could only be used at night. Review of the contractor's schedule comparison revealed several opportunities for time savings with precast concrete elements. For example, curing the grout in different locations could be simultaneous rather than sequential. In addition, the curing times needed before proceeding to the next activity were shorter than assumed.

In cast-in-place concrete construction, cofferdams cannot be removed until after the crossbeams have cured, forms have been stripped, shoring has been removed, and the final grading and embankment protection have been completed. With precast concrete, the final grading could begin immediately after the columns were set and the column-to-shaft closure was cast and cured. Precast concrete construction also afforded better access for grading, compacting, and placement of riprap without the crossbeam in place. If necessary, the precast concrete portion of the crossbeam could be set, filled, and cured outside of the fish window after removal of the cofferdam.

Using cast-in-place concrete would have required a crane for multiple activities. Any activities requiring boom lengths in excess of 25 ft (7.6 m) would have required the work to be performed at night. Such activities include lifting and placing column reinforcement cages, lifting and placing column forms, casting columns, placing shoring and scaffolding, placing crossbeam formwork, and placing crossbeam reinforcement, ducts, and strand.

The full-depth precast concrete deck panels were erected on top of the steel plate girders. Due to proximity to the runway, this work was done at night to avoid interfering with operations at Renton Municipal Airport. Courtesy of Ramón Mariano, Jr.

The contractor ultimately found ways to significantly shorten the schedule, which was fortunate, because new soils information led to significantly deeper drilled shaft foundations than anticipated. Part of the crossbeam construction took place after the fish window, which was made possible by the use of precast concrete bridge elements.

Lessons learned

Precast concrete columns

A reduction in section was provided at the bottom of the column where it transitioned from circular to octagonal. The reduced section was detailed to be held a few inches above the top of the shaft to better control the plastic overstrength forces being transferred into the adjacent capacity-protected elements. The top of the shaft was located approximately 9 ft (2.7 m) below the mud line while the column-to-shaft transition section was cast. As a result, controlling the top-of-shaft elevation was difficult and the contractor was instructed to place concrete up to the bottom of the circular column section to avoid stopping the top of the shaft too low and potentially exposing the octagonal section of the column with minimal cover to soil.

Crossbeams

The recess in the soffit of the precast concrete portion of the crossbeam was rectangular rather than circular to match the shape of the column. As a result, a special soffit form was required to facilitate grouting the ducts and interface between column and crossbeam.

Precast concrete shop drawings needed to include the adjacent components to avoid interferences. Bearing anchorage assemblies were being set into the cast-in-place concrete infill portion of the crossbeams. The bearing anchorage assemblies shown in the contract plans were not provided to the precaster, and the precaster developed shop drawings without knowing about the bearing anchorage assemblies. As a result, some of the bars projecting out of the precast concrete portion of the crossbeam into the cast-in-place concrete infill had to be adjusted to avoid the bearing anchorage assemblies.

Full-depth precast concrete deck panels

The panels were originally detailed with three 1 in. (25 mm) diameter pour holes over each hidden pocket. They were modified to include one 4 in. (100 mm) pour hole and one 1 in. air release over each hidden pocket. The pour holes were located on the high end of each panel.

The drawings called for ultra-high-performance concrete to fill the annulus between the top of the girder flange

Panel sequencing plan

N

Panel number:	P 1	P 2	P 3	P 4	P 5	P 6	P 7	P 8	P 9	P 10	P 11	P 12	P 13	P 14	P 15	P 16	P 17	P 18	P 19	P 20	P 21	P 22	P 23	P 24	P 25	P 26	P 27	P 28	P 29	P 30
Sequence number:	1	2	3	30	29	28	27	18	17	16	15	14	13	12	7	8	9	10	11	19	20	21	22	23	24	25	26	4	5	6
Load number:	1	1	1	4	4	4	4	3	3	3	3	2	2	2	2	2	2	2	2	3	3	3	3	4	4	4	4	1	1	1

Marked end facing south (P 1–P 15) | Marked end facing north (P 16–P 30)

*Note: Verify large vent tube is facing in the up-slope direction

The end panels were erected first to serve as counterweights to keep the girders on their bearings while the other panels were erected. This work took four nights to complete; however, the contractor later estimated that it could have been completed in two nights. Courtesy of Greg Banks.

and the panel soffit with placement through the vent holes, leaving the vent holes filled. The contractor had not accounted for the time required for the ultra-high-performance concrete to consolidate, set, and vent, resulting in recesses that significantly exceeded the allowable relief. Highlighting the material behavior in the specifications and requiring the contractor to provide for overfilling or overpressure to compensate would minimize the need for patching.

The bridge profile has approximately 4 ft (1.2 m) of grade difference between the midpoint and ends. A placement sequence for the ultra-high-performance concrete was specified relative to placement of the panels, grouting of the transverse joints, and posttensioning of the panels, resulting in a continuous ultra-high-performance concrete haunch connecting the haunches and hidden pockets along the length of the girder. The contractor expressed concerns with controlling formwork leaks with 4 ft of head and requested to partition the haunches with stay-in-place bulkheads. The potential benefits of intermittent bulkheads should be considered in design, and specifications should include requirements for the contractor's formwork submittal to account for pressure and include a layout for proposed bulkheads.

The continuous transverse shear key at each panel edge was filled with a nonshrink grout to aid in shear transfer between panels. To facilitate panel placement and prevent leaking during grout placement, a ½ in. (13 mm) backer rod below the shear key was specified. The contractor placed the backer rod higher than specified, creating an eccentricity in the posttensioning of the panel. This unintended eccentricity would result in tension at the top of the joint under service loads. As a result, the contractor was required to push down the backer rod within the tensile zones. Recommendations for future projects would be to provide a placement tolerance in the design or use a formwork detail that provides for full panel depth at the joint.

The longitudinal panel reinforcement and posttensioning ducts were spliced at each transverse panel joint. Rectangular blockouts were provided in the panels to facilitate splicing. The required blockout size can be highly dependent on the coupler product used. If contract requirements do not allow the designer to select specific duct splices or mechanical couplers, specifications need to include provisions for the shop drawing submittal to include mechanical coupler and duct splice details that work with the plan blockout size or submit modifications for review.

Partial-depth blockouts were provided at the lift-loop and leveling-bolt locations. The blockouts were originally to be patched with a nonshrink grout. The durability of the exposed patches was of concern, especially with the potential for water intrusion in the tensile zones. With the bridge functioning as a taxiway for the adjacent runway, there was concern regarding debris from a failed patch to be carried onto the runway. As a result, patching material and surface preparation requirements were modified to follow FAA recommendations for runway patching using a low-modulus epoxy mortar. In addition, design documents called for a high-molecular-weight methacrylate to be applied across the entire deck. Recommendations for future projects would be to consider detailing blockouts in full-depth concrete panel decks with a mechanical interface or anchorage and to avoid details that rely on bond.

Conclusion

The Boeing North Bridge was completed in November 2014, just before Boeing increased production. The project represents a successful deployment of accelerated bridge construction methods using precast concrete in high-seismic regions. The project showed that conventional construction practices may need to be modified to better suit accelerated bridge construction. For example, accelerated bridge construction may take some work out from the general contractor's control and require fewer workers.

In evaluation of accelerated bridge construction methods, the temporary works and equipment cost differentials between cast-in-place concrete construction methods and the use of prefabricated bridge elements should be considered. On the Boeing North Bridge, precast concrete elements required larger cranes than would have been needed for cast-in-place concrete. Use of larger cranes required work to be done at night to avoid interfering with the operations of the airport. Ultimately, these operational restrictions proved comparable to those that would be encountered with cast-in-place concrete.

The use of precast concrete substructure elements and full-depth precast concrete deck panels is relatively new to the Pacific Northwest. On future contracts using precast concrete bridge elements in accelerated bridge construction, it may be worthwhile to require a kickoff meeting with the contractor to discuss construction methods and tolerances.

Acknowledgment

All precast concrete components were fabricated by Concrete Technology Corp. in Tacoma, Wash. AMEC Environmental and Infrastructure, Inc., provided environmental consulting services.

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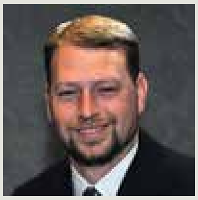
About the authors

Gregory A. Banks was the lead design engineer for the Boeing North Bridge for BergerABAM. He has over ten years of experience in bridge and transportation projects and is a consulting member of PCI's committee on bridges. His expertise includes prestressed concrete bridge design, accelerated bridge construction, segmental bridge design, and seismic design of bridges.



Myles Parrish was the project engineer for the Boeing North Bridge for BergerABAM. He has more than 15 years of experience in the design of bridges and buildings.

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Abstract

The Boeing North Bridge is a multispan girder bridge spanning the Cedar River in Renton, Wash. All aircraft assembled in Boeing's factory are towed over the bridge to Renton Municipal Airport, where they undergo final inspection before takeoff. The design and construction schedule for the bridge was accelerated to accommodate Boeing's increased production. To meet the constraints imposed by environmental regulations, winter construction, airport operations, and the surrounding community, the new bridge was designed using precast concrete columns, partially precast concrete crossbeams, and full-depth precast concrete deck panels. Due to the high seismicity of the site, the connections between the substructure elements had to be seismic resisting. This paper summarizes the details of the precast concrete bridge elements and the construction methods used.

Keywords

Accelerated bridge construction, connection, seismic design, ultra-high-performance concrete.

Reader comments

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