I-205 Columbia River Bridge — Design and Construction Highlights

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Giant prestressed concrete box girders, built with a combination of precast and cast-in-place construction are rapidly spanning the Columbia River (Fig. 1) near Portland, Oregon, to complete the first wave in a new generation of large-scale, segmental concrete bridges in North America.

The Columbia River Bridge consists essentially of two parts — a North and a South Channel superstructure. Twin trapezoidal box girder structures, spanning 7500 ft (2287 m), will carry four lanes of traffic over the North Channel.

Nearly 600 precast concrete segments and 184 cast-in-place segments make up the superstructure. The precast segments, weighing up to 180 tons (163 metric tons), are 71 ft (22 m) wide and 12 to 17 ft (3.7 to 5.2 m) deep. Segments 71 ft (22 m) wide, up to 32 ft (10 m) deep and 16 ft (5 m) long were cast in place for the 600-ft (183 m) main span over the navigation channel. The above dimensions and variations exemplify the complexity of the job.

The unusual combination of cast-in-place and precast concrete construction involves using four hydraulically controlled travelers, each able to support 60 tons (54 metric tons) of formwork for cast-in-place cantilevered segments, and four pairs of 100-ton (91 metric tons) hoists that lift and position precast segments up to 150 ft (46 m) above the Columbia River.

NOTE: This paper is based on a presentation given at the Segmental Concrete Bridge Conference in Kansas City, Missouri, March 9-10, 1982. The Conference was sponsored by the Associated Reinforcing Bar Producers—CRSI, Federal Highway Administration, Portland Cement Association, Post-Tensioning Institute, and Prestressed Concrete Institute.

Presents the major design and construction highlights of the $130 million North Channel prestressed concrete structures of the Columbia River Bridge, built using a combination of precast and cast-in-place segmental construction. Problem areas encountered during construction are discussed and recommendations are given to avoid such problems in the future.

As part of Interstate 205, the $175-million Glenn L. Jackson Columbia River crossing will be a major link in a 40-mile (64 km) circumferential system that bypasses Vancouver, Washington and Portland, Oregon. Construction began in 1977 and plans call for the bridge to be in service in late 1982.

Each 68-ft (21 m) bridge deck will carry four traffic lanes and two 10-ft (3.05 m) shoulders. A precast concrete slab will close the 12-ft (3.66 m) space between the decks to provide a bicycle and pedestrian pass.

Fig. 1. Columbia River Bridge nearing completion. This view is looking south towards Oregon across the North Channel structures, Government Island and the South Channel structures.

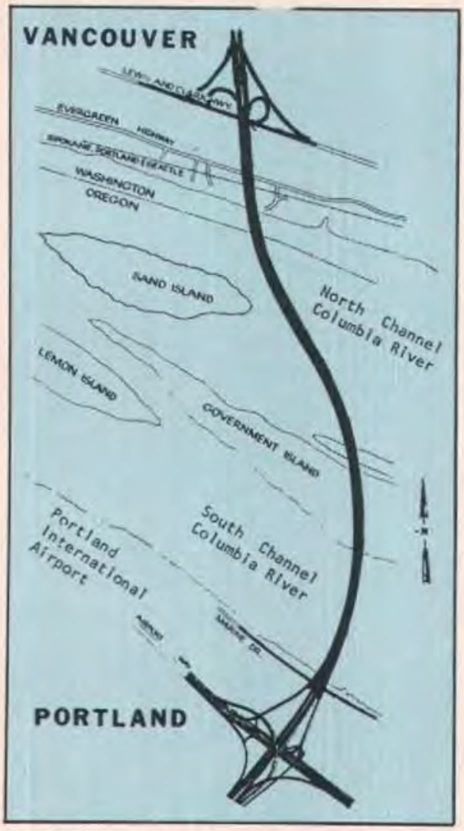


Fig. 2. Location of Columbia River I-205 Bridge.

The decision to build the bridge with prestressed concrete box girders instead of steel followed a comprehensive study for the Oregon Department of Transportation (ODOT) by the consulting engineering firm Sverdrup & Parcel and Associates.

This firm also handled the original design of the north approach spans and the main north navigation channel substructure and superstructure. In addition, the firm is construction consultant. The ODOT designed the project's shorter crossing of a channel on the south side of the river. Designs underwent modifications after contract awards were made.

The design of the bridge presented an intricate challenge. Besides being visually pleasing, minimizing environmental impact and meeting alignment objectives, the crossing had to be sufficiently high on its north, or Washington, main channel side to clear oceangoing ships. On the south side, however, it must descend sharply to pass under airplane approach paths for nearby Portland International Airport. It also has to touch down and connect with an 1100-ft (335 m) road on a small island in the river before proceeding over the south channel to the Oregon shore (Fig. 2).

To meet these demands, the bridge extends over the river in a sweeping curve from 48 to 150 ft (15 to 46 m) above the water.

The $130 million North Channel structures will be completed under four separate contracts with several design variations and construction techniques being utilized. Fig. 3 is a sketch showing the breakdown of the various construction methods.

Four major I-205 contracts have been awarded by ODOT: the north channel substructure to Riedel International, Inc., Alaska constructors and General Construction Co., for $30 million; the north main channel superstructure to a joint venture of S. J. Groves & Sons Co. and Guy F. Atkinson Co., for $71.6 million; the south channel crossing to Riedel International, Inc., for $35.7 million; and a $3.1 million contract to Eisenhour Construction for placing a 2-in. (51 mm) latex modified concrete wearing surface on the North Channel structure.

The Washington Department of Transportation awarded the north approach spans contract to Peter Kiewit Sons' Co., for $12.8 million.

To encourage bids and take advantage of contractor capabilities, bid documents contained value engineering incentive clauses.

The redesign of the longer North Channel superstructure by T. Y. Lin

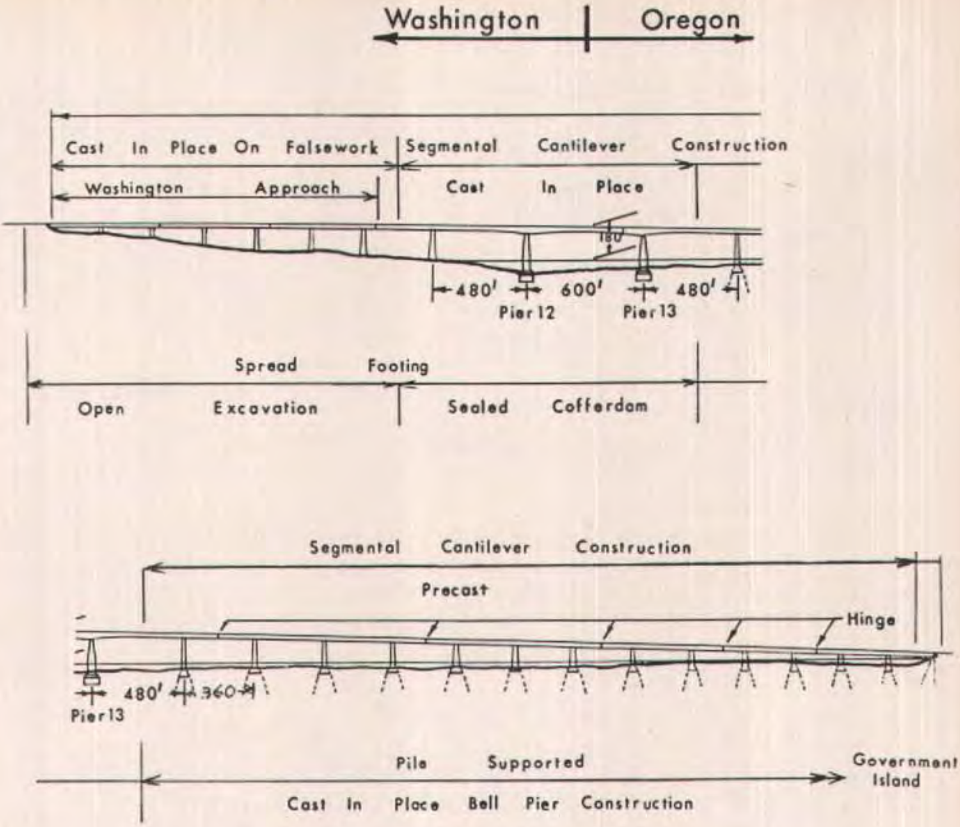


Fig. 3. Constructed under four separate contracts a variety of construction techniques were used in building the 7500-ft (2287 m) long crossing of the North Channel of the Columbia River Bridge.

International, replaced the original three-cell girder with a two-cell version, cutting reinforcing bars, concrete and post-tensioning requirements. A redesign of the north approach spans by DRC Consultants, Inc. increased girder depths, extended span lengths and reduced the number of piers from ten to seven per deck.

After briefly reviewing the pier construction and the Washington approach spans, the rest of the article is devoted mainly to describing the design and construction aspects of the North Channel superstructure. Highlighted are the value engineering redesign of the superstructure, the production and erection of the precast segments, geometry control, duct alignment, and span closures.

Briefly mentioned is the on-going program to monitor strains and relative concrete temperatures at various locations throughout the structure. Lastly, the construction problems encountered are discussed and recommendations are given to avoid such pitfalls in the future.

PIER CONSTRUCTION CONTRACT

The first contract for construction of the river piers was awarded in 1977. The contractor utilized a welded steel form which doubled as a cofferdam to construct 26 pile-supported piers (Fig. 4).

After excavation, permanent steel support piles were driven and cut off under water. The bell shaped welded steel form was lowered into place over the 14-in. (356 mm) steel "H" piles. A 9 to 12-ft (2.7 to 3.7 m) seal placed by the tremie method allowed dewatering the form. Reinforcing steel was then placed, the form filled with concrete and the pier base completed.

The two piers supporting the main 600-ft (183 m) span over the navigation channel were constructed with cofferdams utilizing 100-ft (30.5 m) long, 22-in. (305 mm) deep interlocking steel "H" sheet piles. The critical operation in constructing the base for the piers was pouring of 9700 and 11,200 cu yd (7420 and 8568 m^3) seals by the tremie method.

This was accomplished in around the clock pours using a 200 cu yd (153 m^3) per hour floating batch plant (Fig. 5). Piers were completed to about one-half their final height above the water under this contract.

WASHINGTON APPROACH CONTRACT

In constructing the 1700-ft (519 m) Washington shore portion of the structures, the contractor cast trapezoidal box shaped segments up to 71 ft (21.7 m) long in-place on falsework (Fig. 6).

When segments for a span were completed plus one segment in the next

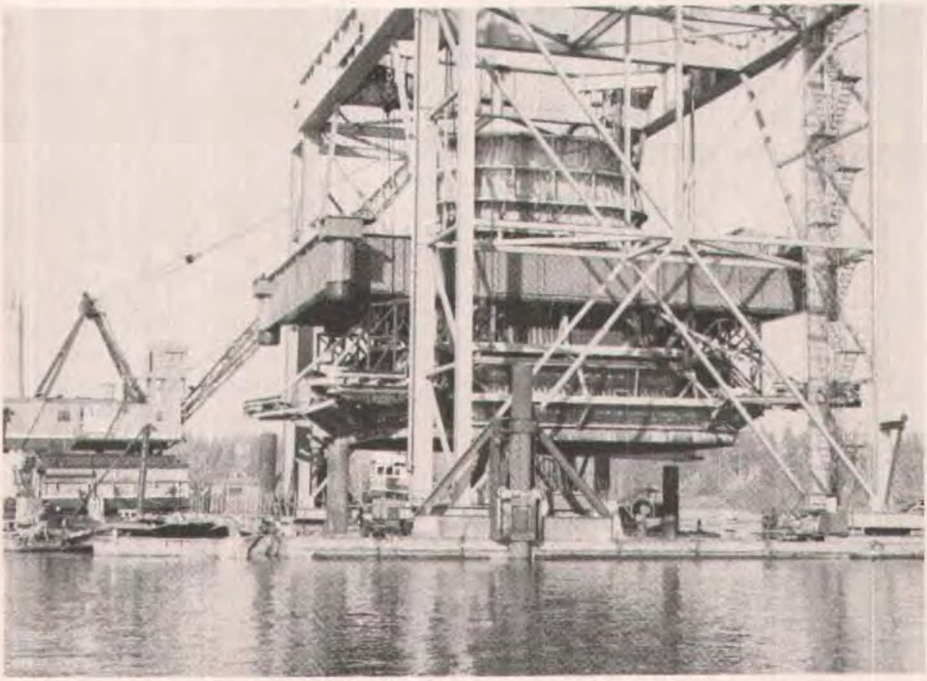


Fig. 4. Preparing to lower the 58 x 76 x 63-ft (17.7 x 23.2 x 19.2 m) high-welded bell-shaped steel form.

Fig. 5. Placing concrete by the tremie method for the 50 x 150 x 39-ft (15.2 x 45.7 x 11.9 m) deep seal at Pier 13. Placement of the concrete for the seal was completed in 55 hours.

Fig. 6. The Washington Approach spans were constructed by pouring concrete in-place for 71-ft (21.7 m) segments. Falsework up to 120 ft (36.6 m) high supported the spans.

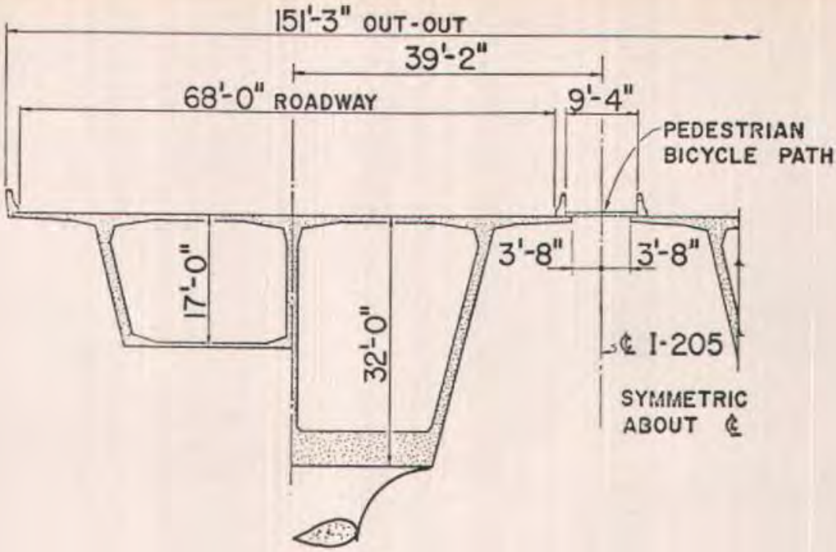


Fig. 7. Cross sections through the value engineering redesign of 600-ft (183 m) main span with the 32-ft (9.76 m) depth at the pier and 17-ft (5.18 m) depth at the midspan.

span, profiled post-tensioning tendons were stressed, freeing supporting falsework to be moved ahead. Post-tensioning tendons in adjacent spans lapped past the piers. The overlapping of the tendons of adjacent spans at the pier established continuity through the multispan units.

Modular sections of falsework were stacked to create supporting bents up to 120 ft (36.6 m) high (Fig. 6). The span by span stressing procedure significantly reduced the units of falsework from that normally required for continuous span construction.

NORTH CHANNEL SUPERSTRUCTURE CONTRACT

The contractor on the North Channel section opted for a combination of cast-in-place and precast segments. Though designed to employ precast segments, the choice of precast or cast-in-place segments for the balanced cantilever segmental construction was given to the contractor.

Trapezoidal box girder segments for the main 600-ft (183 m) span over the navigation channel were cast in place. Transition from the use of cast-in-place segments to precast segments was made at the balance point in the 480-ft (146 m) span adjacent to the main span of the bridge.

The span configuration was established to permit balanced cantilever construction without the need for temporary supporting falsework except at the end spans. The piers and the superstructure to pier connections were designed to allow an unbalance during cantilever construction of one segment plus the lifting equipment.

Joints to accommodate expansion were called for at the $^{3}/_{10}$ths point in the expansion spans. A hinge at the $^{3}/_{10}$th point required locking the hinge and cantilevering through the hinge to the closure at the balance point.

Fig. 8. The 40 x 40 x 8-ft (12.2 x 12.2 x 2.44 m) test section was constructed prior to casting segments.

Value Engineering Redesign

Under the value engineering clause in the contract the contractor proposed and the state accepted a redesign of the superstructure from a three-cell box to a two-cell box (Fig. 7) and shifting of the expansion hinges from the $\frac{3}{10}$th point in the expansion spans to the cantilever balance points.

Particularly interesting and controversial was the relocation of the hinge. Historically, the problem with midspan hinges has been excess deflection at the hinge with time due to creep of the concrete.

Several design features were considered to alleviate this problem. The design finally accepted calls for sufficient extra post-tensioning in the hinge spans to create a uniform stress block. Also, extra ducts were provided to allow for additional post-tensioning in the top slab if needed.

The discussion below will concentrate on the construction using precast concrete segments.

Casting Precast Segments

A demonstration segment was required prior to starting full-scale production of segments for the project. The demonstration segment could either be the first segment cast for incorporation in the project or a facsimile mockup.

A 40 x 40 x 8-ft (12.2 x 12.2 x 2.44 m) deep mockup was constructed to demonstrate the adequacy of various tendon anchors, anchor block reinforcement systems and construction techniques (Fig. 8). As a result of mockup testing, the anchor system previously unused in Oregon was approved and changes in the proposed stressing procedures and anchor block reinforcement were made.

The 592 precast segments for the project, cast in a yard 5 miles (8 km) downstream from the bridge site are 71 ft (21.7 m) wide, 12 to 17 ft (3.66 to 5.18 m) deep, 7½ to 12 ft (2.29 to 3.66 m) long and weigh up to 180 tons (163 metric tons). The segments were match-cast by the short-line method. Four sets of forms were used in the manufacture of the segments (Fig. 9).

After a segment was completed it was rolled ahead and used as a bulkhead against which the next segment was cast. Prior to placing the concrete for the next segment the completed seg-

Fig. 9. Four casting machines were used in manufacture of precast segments. Tower in background is for survey control. In foreground are the reinforcing steel templates.

ment was rotated vertically and horizontally to establish the proper geometric relationship between the segments.

Before a segment could be moved out of the form, partial stressing of transverse tendons was required to avoid tension in the top slab. Initial stressing of the transverse tendons was limited to 20 percent of the final stress to reduce the possibility of distortion of the completed segment prior to making the match-cast joint.

Punch-marked angles to be used as reference points were cast in each segment on the bridge centerline and 28 ft (8.54 m) left and right of the centerline at each segment face. Prior to separating the segments in the casting machine, distances between the reference points were measured and relative elevations taken to determine the "as cast" geometric relationships between the segments. This information was used to adjust subsequent segment geometry, to aim the cantilever and check the alignment as erection of the cantilever progressed.

Fig. 10 shows 12 and 17 ft (3.66 and 5.18 m) deep segments in the storage yard.

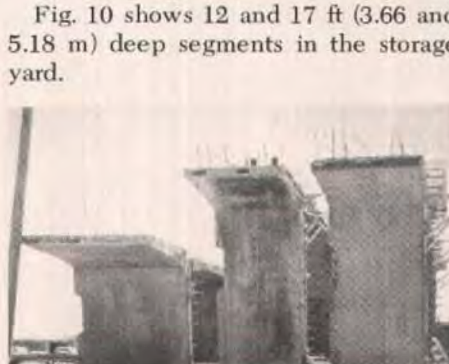


Fig. 10. 12 and 17-ft (3.66 and 5.18 m) deep segments shown in storage. All three precast segments shown are elements which will be adjacent to pier tables.

Fig. 11. Lifting the first segment with closure forms attached.

As needed segments were barged to the site two at a time. Access to all spans by water made the use of winch and beam lifting equipment practical.

Erection of Precast Segments

A 40-ft (12.2 m) section of the superstructure centered over the columns referred to as the pier table, was cast in place. The first segments were lifted on each side of the pier table and positioned with a 1 ft 9 in. (0.534 m) gap between the segment and the pier table (Figs. 11 and 12). The segments were then "aimed" using "as cast" measurements taken in the yard. Concrete was then placed in the gap. The 1 ft 9 in. (0.534 m) gap was reduced to 4 in. (102 mm) on later cantilevers.

A problem experienced at a number of piers was getting the alignment of the ducts in the segment to match the duct alignment in the pier table. Even the use of full-sized templates did not totally solve the problem. Extra reinforcement to control bursting forces caused by duct misalignment was added to the closure section. Sometimes the effect of misalignment extended beyond the closure into the segment or pier table or reinforcement added was insufficient and spalling occurred when permanent tendons were stressed.

Fig. 12. Ready to make the 1 ft 9 in. (0.534 m) closure pour between the first segment and the pier table.

When the concrete strength in the closure between the pier table and the segment reached 3500 psi (24 MPa), usually in less than 48 hours, tendons were fully stressed, lifters were moved ahead and erection of the match-cast segments proceeded (Fig. 13).

The match-cast joint faces were coated with epoxy (Fig. 14) and the segments drawn together with temporary rods. A temporary force sufficient to produce a compression of 50 psi (0.345 MPa) over the joint was applied. Failure of the concrete under the temporary stress rod bearing plates occurred in the early segments (Fig. 15). The failures were eliminated with the addition of cushioning under the plate to better distribute the load and the addition of spiral reinforcement. When

Fig. 13. Segment lifted is ready for application of epoxy to seal the match-cast joint. Note the large single key.

Fig. 14. Application of epoxy to match-cast face.

balancing segments were in place on each side of the pier, permanent post-tensioning tendons were installed and stressed and the rods removed.

Single large heavily reinforced keys were provided in each web for aligning the segments and transfer of vertical load during erection (Fig. 13). In the completed cantilevers a combination of the large keys, draped tendons passing through the joint and the epoxy is depended upon to carry the vertical load across the joint.

Fig. 15. Crushing of concrete behind the bearing plate of one of twelve temporary stress rods used to apply compression to match-cast joints. Rods are removed when permanent tendons have been stressed.

Fig. 16. To develop additional load transfer at a match-cast joint in which the epoxy did not set, 4-in. (102 mm) diameter holes were cored into the joint and filled with spiral reinforced high strength mortar.

Fig. 17. Blockouts in the adjacent segments opposite the draped tendon achorage, when filled with concrete, provided additional shear keys.

About the time casting of segments started, reports of problems with similar joints in another segmental structure were received. A redesign of the joint and the use of multiple keys was considered.

Much of each web face was occupied by anchorages for the draped tendon leaving little space for additional keys. Use of smaller unreinforced keys was not favored and the decision was made to adjusts and increase the mild steel reinforcement but keep the design for the single large key.

The epoxy in the first joint failed to set properly. By the time the problem was discovered, disassembly was not practical. To eliminate the reliance on the epoxy, additional load transfer capacity was developed by filling cored 4-in. (102 mm) diameter holes in the joint. Spiral reinforcement consisting of coil springs was placed in the cored holes and the holes filled with high strength mortar (Fig. 16).

Also, opposite the blockout for the lower tendon anchorages, concrete was broken out to create a matching recess. Reinforcing steel was placed in the void area and the void filled with concrete to create an additional key.

We were so satisfied with the filled key that voids were routinely cast in adjacent segments opposite draped tendon anchorages and filled with reinforced concrete after the draped tendons were stressed (Fig. 17).

Alignment Adjustment

As erection progressed, alignment of the cantilevers was compared with the projection developed from the as-cast measurements. Although horizontal alignment was no problem, vertical alignment was. In general, as erection progressed the cantilevers were high. Shimming was allowed by the contract; however, the thickness of the shims was limited to ⅛ in. (3.2 mm). The shims were tried (Fig. 18) but they had little if any effect. In addition to the problem with vertical alignment, twist developed in some cantilevers.

The only effective correction found for both twist and vertical misalignment was construction of a wet joint between segments. The corrective wet joint was constructed by blocking the joint open 2 to 4 in. (51 to 102 mm) and re-aiming the segment being added (Fig. 19). The joint was then filled with mortar. Some cantilevers required several wet joints,

Fig. 18. Expanded metal shims ⅛ in. (3.2 mm) thick being placed in joint in attempt to adjust alignment. In shimming a joint, approximately 1000 sq in. (0.645 m^2) of shims were placed in the top or bottom slab at the webs.

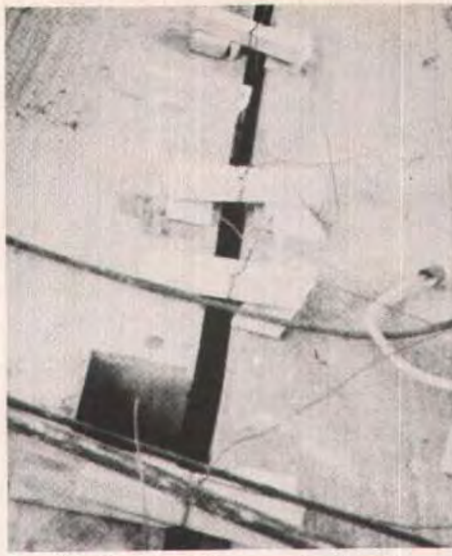


Fig. 19. Ready to place mortar for 2-in. (51 mm) wet joint to adjust alignment and for twist.

Fig. 20. Weight to produce an unbalancing load of up to 100 kips (445 kN) was placed on the end of the cantilever to adjust alignment.

Fig. 21. Forms in place for closure pour. The diagonal stress rod in the bottom left of the picture was used for alignment adjustment and maintaining alignment during the closure pour.

although others were completed with no wet joints.

Some adjustment of vertical alignment was possible by weighting cantilevers (Fig. 20) or by jacking one cantilever against the other at the closure (Fig. 21). Weighting and jacking was limited to that which would create an unbalancing load on the pier equal to 100 kips (445 kN) applied at the end of the cantilever. A 100-kip (445 kN) weight on one cantilever would force that cantilever down about 1¼ in. (32 mm) (more or less depending on the cantilever length and pier stiffness) but would raise the end of the cantilever on the other side of the pier about ⅝ to ¾ in. (16 to 19 mm). Thus, if both cantilevers out from a pier were high, little could be done to correct the condition.

The 8-ft (2.44 m) gap between cantilevers also allowed for minor alignment adjustment (Fig. 21). Specifications required the vertical alignment at the ends of the cantilevers to differ by no more than ⅝ in. (16 mm).

Span Closures

In closing the gaps between cantilevers in the two or three span units, alignment across all gaps within a unit and across the hinges would be considered and initially adjusted prior to making any closure within a unit. All of the fixed joints within two adjacent units were completed prior to constructing the hinge.

Procedures for making the closures between cantilevers were developed to minimize tension and prevent cracking. The bottom slab was of particular concern. The procedure called for locking the cantilevers to prevent differential deflection under the weight of closure concrete. Struts were installed between the bottom slab and prestressing strand

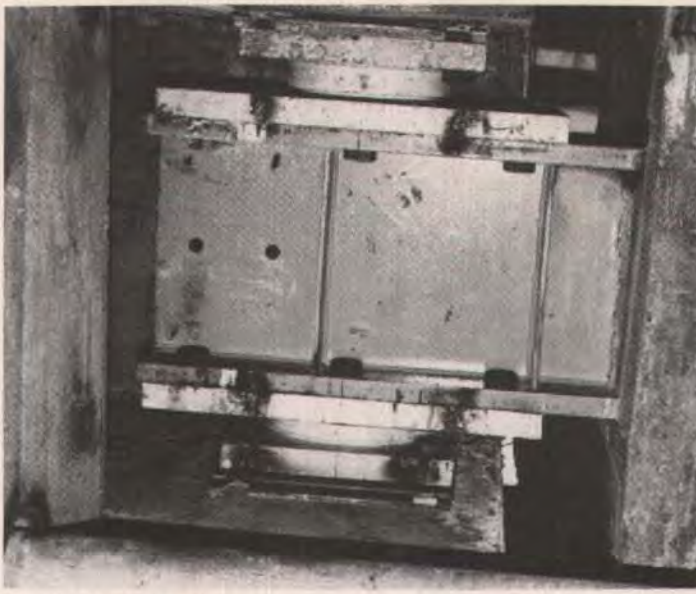


Fig. 22. Hinges with opposing pot bearings at midspan.

installed and stressed putting the struts in compression. As soon as the concrete set, additional bottom slab continuity tendons were stressed.

During the summer months closure pours were made in the evening so that thermal shortening of the top deck would tend to raise the cantilever offsetting any deflection from the weight of the closure concrete. The following morning, the struts were cut and additional strands were stressed.

Apparently the procedure was effective because no cracking has so far been detected in the closure sections.

With the hinges at the balance points in a span there will be no dead load across the joint so thermal, wind and traffic induced loads may reverse the direction of load transfer through the hinge. This condition caused complex design details and difficult fabrication and construction.

After considering several load transfer systems, a design utilizing opposing pot bearings acting on a protruding hinge beam was finalized (Fig. 22). Since pot bearings were working in opposition to each other and it was deemed desirable to maintain both pot bearings in compression at all times, fabrication and setting tolerances for bearing and bearing beams were very fine.

Duct Alignment

A continuing problem was that of maintaining duct alignment and keeping the ducts open especially the bottom slab (continuity tendon) ducts. Offset of ducts at the joints were common and interfered with the installation of tendons. The problem was reduced by using larger ducts and a semi-rigid plastic sleeve through the match-cast joint during concrete placement.

Cross-over during grouting resulted in some duct blockages. Cross-over was a particular problem in hinge spans where installation of tendons which anchored in the hinge block was delayed until after grouting of all other tendons in the cantilever had been completed.

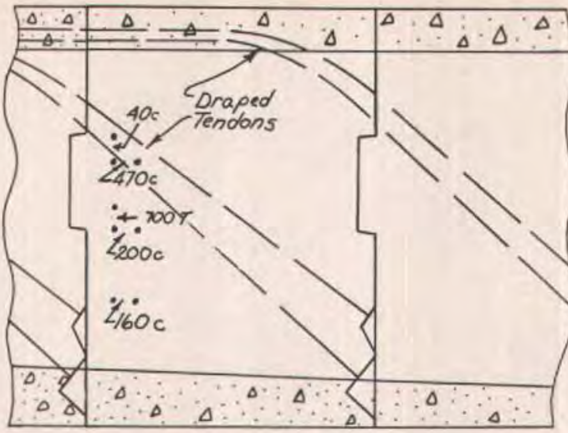


Fig. 23. Strain measurements at a typical alignment key. Note that figures are strains in 1 x 10^{-6} in./in.

Even relatively small amounts of grout in the empty duct made installation of tendons difficult especially when cross-over occurred at more than one joint. At the wet joints where alignment adjustments were made, blockages occurred when inflated tubes used to establish the opening through the joint collapsed. Use of high pressure water jets was effective at some locations in opening the ducts but at other locations it was necessary to expose the duct by breaking out the concrete around the duct so the blockage could be chipped out.

Strain and Temperature Monitoring

An on-going program of monitoring strain and relative concrete temperatures at various locations throughout the structure is being carried out. Strain is being measured with a Whittemore strain gage and by embedded electrical resistance type strain meters. Temperature monitoring is with the resistance strain meters and thermal couples embedded at various depths in different concrete elements.

The purpose of collecting the data is to verify the correctness of various assumptions made in the design of large box girder structures. For example, at the time of design and later during the value engineering redesign there was considerable discussion as to the temperature differential which should be assumed within the box section in designing for zero tension.

To date little effort has been made to analyze the information gathered so other than a few very spotty observations there is little to report. One observation which may be of interest is the extremely high strains recorded in the area of the shear keys.

Measurement of strain in the area of the alignment key indicates up to 700 micro in. per in. (Fig. 23). Assuming a modulus of elasticity of 4 x 10^6 in the concrete and 27 x 10^6 for the reinforcing steel, the corresponding tension in these areas would be 2800 psi (19 MPa) in the concrete and about 19,000 psi (131 MPa) in the steel.

Figs. 24 through 27 show various stages of the precast segmental construction. Figs. 28 through 31 show the cast-in-place cantilever construction.

Fig. 24. Precast segment being lifted into place (North Channel section).

Fig. 25. Precast segment on verge of being joined to superstructure (North Channel section).

Fig. 26. Construction progress showing the cast-in-place pier tables and the advancing precast prestressed superstructure (North Channel section).

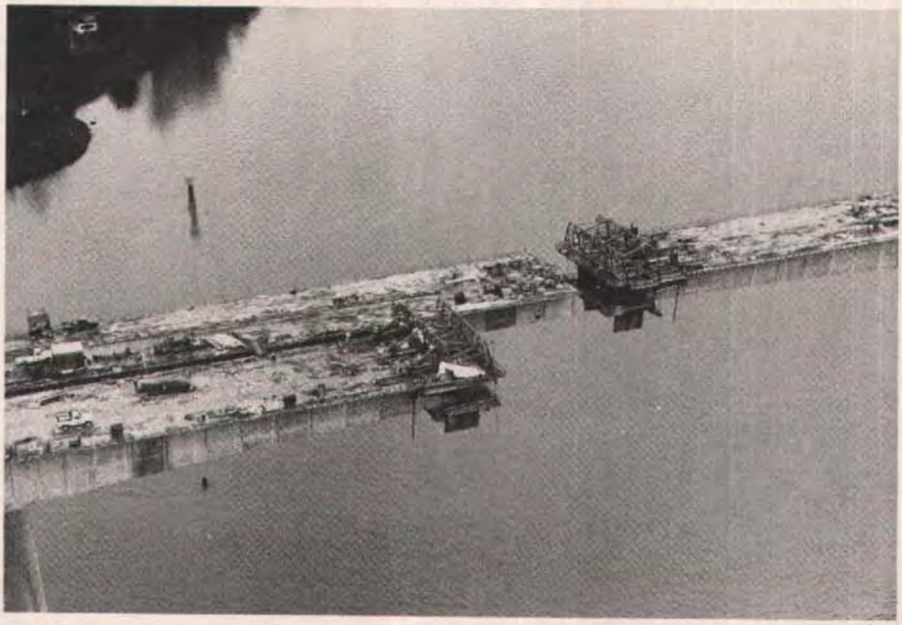
Fig. 27. Overall view of precast segmental construction (North Channel section).

Fig. 28. Nearing completion of the 480-ft (146 m) Span 11 adjacent to the main 600-ft (183 m) span.

Fig. 29. The 296-ft (90 m) cantilever out from Pier 12 over the main navigation channel (northbound structures).

Fig. 30. The 176-ft (54 m) cantilevers on left (foreground) are made up of precast segments. On the right is the cast-in-place segmental cantilever from Pier 13.

Fig. 31. One more segment to pour for the northbound structure then the closure for the 600-ft (183 m) main span over the navigation channel.



CONCLUSIONS AND RECOMMENDATIONS

Long span concrete structures can be constructed efficiently and economically with either precast or cast-in-place segmental cantilever construction. Each method has its advantages and both techniques have areas which can produce potential problems. While I cannot pinpoint the exact cause of some of the problems we experienced on the I-205 Columbia River Bridge (Fig. 32), I can make some recommendations which I believe will minimize these problems.

1. Provide mild steel containment reinforcement normal to the axis of the post-tensioning tendons. In particular, use vertical ties between the top and bottom reinforcing mats at segment joints.

2. Pay particular attention to duct alignment at joints during casting of segments.

3. Minimize the dependence on epoxy for load transfer.
   (a) Avoid use of shims.
   (b) Apply sufficient force to close the joints.
   (c) Develop supplemental keys.

4. If shims are used, provide sufficient shim area to avoid excessive concrete stress under the shims.

5. Use pre-bent ducts when curving of tendons is necessary.

6. Use heavy ducts having a minimum wall thickness of 0.030 in. (0.76 mm).

7. Spread the ducts as much as possible.

8. Seal around the ducts at time of erection.

9. Do not become overly concerned in making fine alignment adjustments at the time segments are cast but do make a careful measurement to determine the as-cast geometry of the segments and the relative geometry of the adjacent segment.

10. Avoid distortion of the segment between the time it is cast and when the next segment is cast.

11. Avoid accelerated curing. If not feasible, delay the application of heat until the concrete is set.

12. Maintain a uniform internal concrete temperature in the segments especially during the match casting operation.

13. Minimize the temperature differential between the segment in the match-cast position and the segment being cast.

14. Keep the face of the bulkhead smooth. As ducts are dropped, patch the holes in the bulkhead so there is no protrusion.

15. Design a temporary stressing system so that the temporary stress can continue to be applied as long as needed.

16. Arrange the temporary stressing system so that the application of force is in the same area as the permanent post-tensioning.

Many of the problems we encountered are not new and undoubtedly some of the remedies offered have been tried before. Nevertheless, our experiences are shared in the hope that these problem areas will not occur again in future prestressed segmental bridges.

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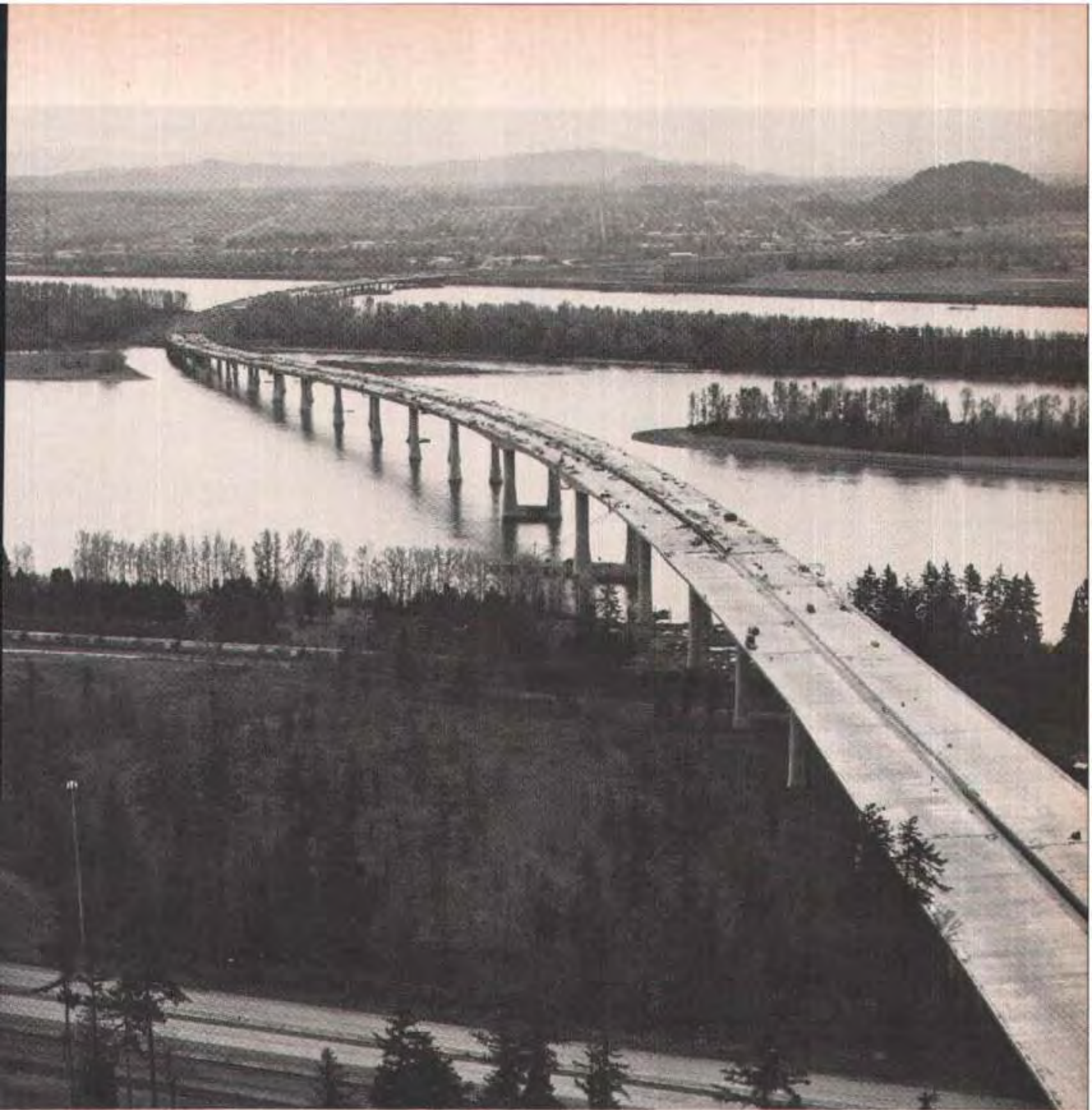

Fig. 32. I-205 Columbia River Bridge nearing completion. Current plans call for the bridge to be in service in late 1982.

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by November 1, 1982.