This article describes the design and construction of the Old 99 (Riverside) Bridge, an 850-ft-long (260 m), 72-ft-wide (22 m), five-span, post-tensioned, spliced-girder bridge spanning over the Skagit River in Washington State. In 2004, this bridge won a PCI Design Award. The bridge's superstructure consists of recently developed Washington State Department of Transportation W95PTG “supergirder” sections. These precast concrete girder sections were transported to a staging area close to the site, where they were spliced into single pieces that produced maximum spans of 180 ft (55 m). After delivery to the site, the post-tensioned girders were erected on top of the piers with no intermediate temporary supports. High-performance concrete (HPC), with design strengths of 7.5 ksi (52 MPa) and 10 ksi (69 MPa), was specified for the cast-in-place splices and precast supergirder segments, respectively. The use of HPC allowed a high initial post-tensioning force to be applied to the precast concrete girders. Special attention was paid to the lateral stability during erection and time-dependent camber of the assembled girders.
The Old 99 (Riverside) Bridge links the twin cities of Mount Vernon and Burlington in northwestern Washington (Fig. 1). The bridge’s superstructure consists of three 180-ft-long (55 m) interior spans and two 150-ft-long (46 m) end spans. These spans are supported by the recently developed Washington State Department of Transportation (WSDOT) W95PTG precast concrete “supergirder” sections, which serve as longitudinal stringers. The superstructure is semi-integral at the abutments and hinged longitudinally at the interior piers. Figure 2 shows plan, elevation, and cross-sectional views of the Old 99 Bridge.

The Skagit Valley is located between the Cascade Mountains (to the east) and the Pacific Ocean (to the west). Strong atmospheric currents produce a prolonged rainy and windy winter season and massive accumulations of snow in the mountains. This flooding results in large quantities of debris drifting past the bridge site, including large logs.

Intermediate, temporary bridge supports, which are often slender, cannot be expected to survive the impact from the debris if they remain in the river channel during the flooding seasons. Moreover, for the protection of migrating salmon, the permissible window for construction in the river is limited to only four months per year, from the beginning of July through the end of October. The regulating environmental agencies do not permit falsework to remain in the river channel outside of this construction window.

Considering these physical and environmental constraints, the use of temporary, intermediate falsework supports between the permanent bridge piers was ruled out, and a design concept based on erection of a single-piece girder was implemented. Using precast concrete hammerheads and drop-in middle segments also require no intermediate falsework; a cost comparison, however, revealed that this alternative would have been more expensive.

BACKGROUND

Although bridges constructed with precast, prestressed concrete girders have a proven economic value, require little or no maintenance, and are aesthetically pleasing, the use of such girders is rare in cases where the maximum span lengths stretch beyond 160 ft (50 m).1

There are many reasons for this anomaly, including material-performance limitations, weight limitations on girder handling and shipping, and a lack of general guidelines for the design and manufacture of longer, precast concrete girders. Through the development of high-performance concrete (HPC),2 more efficient girder shapes,3 and other enhancements, the range of spans for which precast, prestressed concrete girders are used has steadily increased.
PHASE 1

1.1 Construct work trestle, temporary shafts, columns and pier caps.

PHASE 2

2.1 Transport 111 pre-tensioned precast concrete girder segments to the site.
2.2 Assemble the precast concrete girder segments and cast the concrete closure pours.
2.3 Post-tension and grout permanent tendons A, B and C of stage 1.
2.4 Post-tension temporary tendons E in the top flange of girders.

PHASE 3

3.1 Transport spliced and post tensioned girders.
3.2 Erect girders on piers for a given span.
3.3 Brace girders at ends and between supports.
3.4 Construct intermediate diaphragms.
3.5 Release temporary post-tensioning strands in the top flange of the girders.
3.6 Pour lower part of diaphragm over interior piers 2–5 and end diaphragms at piers 1 and 6.

PHASE 4 & 5

4.1 After having erected all girders for all 5 spans, start pouring roadway slab span by span. The minimum concrete strength in the lower part of the diaphragm shall be 3000 psi before pouring any roadway slab.
4.2 Pour upper part of diaphragms over piers 1 through 5.

PHASE 5

5.1 Post-tension and grout stage 2 continuity tendons.
5.2 Construct barrier approach slab and remainder of the bridge. Open bridge to traffic.
One of the major limitations associated with achieving longer spans has been the maximum hauling weight of the precast concrete girders. Presently, Washington has a 200 kip (890 kN) weight limit for these girders. This entails fabricating individual girder segments and transporting the segments to the site, where the segments are then spliced into a single unit by a site assemblage procedure, such as post tensioning.

Some of the advantages of the spliced girder concept are longer spans, fewer substructure units, increased girder spacing, rapid construction, enhanced aesthetics, and reduced superstructure depths. The Old 99 Bridge uses both HPC and spliced-girder technologies to achieve a maximum span of 180 ft (55 m), in a one-piece erection process that requires no intermediate construction falsework.

**DESIGN CONSIDERATIONS**

The 850 ft (260 m) length was dictated by the river hydraulics and the existence of a flood-control dyke at each riverbank. To promote competition among local contractors, two design options were prepared: one with precast, prestressed concrete girders with five continuous spans (150 ft [46 m] end spans and 180 ft [55 m] interior spans) and one with steel plate girders with three continuous spans of 270 ft, 300 ft, and 270 ft (82 m, 90 m, and 82 m). The contractor with the lowest bid, Kiewit Pacific Co., chose the concrete alternative. Its bid was 12% lower than the steel alternative.

**Spliced Girder Concrete Alternative**

The design goal for the concrete alternative was to minimize the number of girders per span and the number of piers in the river. The WSDOT W95PTG girder sections were developed by the Pacific Northwest Prestressed Concrete Institute in collaboration with the WSDOT, with the intention of increasing the span capabilities of precast, prestressed concrete girders.

Transportation constraints and weight limitations prevented the precasting and pretensioning of the girders in a single piece, so the girders had to be produced and transported in segments. Furthermore, due to environmental constraints, the precast concrete girder segments needed to be assembled by post tensioning them at a staging yard.
near the site. After being transported for about 2 miles (3 km), they would be erected on top of the piers, one whole piece at a time, without using any intermediate falsework.

The WSDOT W95PTG "supergirder" sections were selected for the bridge superstructure. Concrete Technology Corp. in Tacoma, Wash., fabricated the girder segments, which were transported to the staging yard and spliced together with Stage 1 post-tensioning.

After the concrete at the closure pours (between adjacent precast segments) achieved its design strength, the spliced girders were transported to a work bridge adjacent to the structure and erected as simple spans. Following erection, and placement and hardening of the deck and the diaphragms over the piers, the spliced girders were integrated into a continuous composite bridge using Stage 2 post-tensioning over the bridge’s entire length.

The bridge substructure consists of 7-ft-diameter columns (2.1 m) resting on top of 9 ft 10 in. diameter shafts (3.0 m) that extend an average of 90 ft (27 m) below the river's mudline. Deep alluvial deposits of soft soil in the riverbed and a seismically sensitive site with liquefaction potential dictated the use of deep foundation drilled shafts. The substructure connections to the superstructure consist of longitudinal hinges connecting the cap beams to the pier diaphragms (Fig. 2). Figure 3 shows a schematic of the construction sequence.

Design Details

An important design parameter of the spliced girder is the concrete strength required at the splice locations; the site-cast concrete strength is usually not as high as the plant-cast girder segments.

In the early stages of design, the 28-day design strengths for the closure pours and precast concrete segments were set at 6.0 ksi (40 MPa) and 8.5 ksi (60 MPa), respectively. A closure pour is the short segment of concrete cast at the staging yard between adjacent precast concrete segments at a splice location. Because of these strength limitations, the simple-span post-tensioning (Stage 1) had to be broken into two substages, namely, "assembly" phase and "after-erection-and-prior-to-slab-pour" phase.

This was due to the stresses induced by handling and hauling the spliced girders at the site. Through the use of HPC, the 28-day design strength for the precast concrete segments was increased to 10 ksi (70 MPa), and the strength for the closure pours concrete was increased to 7.5 ksi (50 MPa). These higher strengths allowed the designers to combine the two post-tensioning substages into one.

For transportation and handling purposes, the precast concrete supergirder segments were pretensioned with twelve 0.6-in.-diameter (15 mm), 270 ksi (1860 MPa) prestressing strands. Stage 1 post-tensioning requirements were based on satisfying various criteria, such as allowable service level stresses, ultimate strength, lateral stability, and camber.

The flowchart in Fig. 4 illustrates the iterative design procedure for determining the Stage 1 post-tensioning requirements. Two of the very important, but less investigated, blocks in this flowchart are the lateral stability and the time-dependent deflection characteristics of the spliced girders.

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A comparative study was performed to investigate the deflection characteristics of these long, spliced supergirders. In the study, the deflection characteristics of a simple beam made up of a standard WSDOT W74G girder with a span of 125 ft (38 m) was compared with a simple beam made up of a W95PTG spliced girder with a span of 180 ft (55 m). Both beams were designed for a typical HS-25 loading.

For the climatic conditions, cement, aggregates, and prestressing and curing practices used in Washington, WSDOT found that the major and significant part of camber is relieved while the girders are simply supported. This condition prevails up to the hardening of the deck. Once the deck has hardened, the resulting composite girder sections become very stiff, and the additional camber required to compensate for all loads applied to the composite girders is comparatively small and practically insignificant.

For this reason, the camber diagrams of the W74G and W95PTG girders shown in Fig. 8 and 9 are for the initial period while the girders are simply supported. Unlike the W74G girder, whose design is governed by service stresses at a few critical locations, the design of a spliced W95PTG girder (that is, the required Stage 1 post tensioning) is governed by camber.

In this example, the Stage 1 post tensioning of the W95PTG girder consisted of three tendons of twenty-two 0.6-in.-diameter (15 mm) strands. As shown, the W95PTG section exhibits a small negative camber by the time the deck concrete has hardened or the superstructure has been transformed to a composite section, even under a high post-tensioning force, unlike the W74G section.

The Stage 2 continuity post tensioning consisted of two tendons. One large tendon consisted of twenty-two 0.6-in.-diameter (15 mm) strands, and one small tendon consisted of four 0.6-in.-diameter (15 mm) strands. Theoretical deflection calculations indicate that the Stage 2 post tensioning would cause an instantaneous upward deflection of approximately 0.5 in. (13 mm) on the composite section. Due to the large number of precast concrete segments that needed to be spliced, unintended tendon deviation angles at the closure pours were expected.

The main purpose of the small continuity tendon was to serve as a reserve capacity source and would be stressed only if the frictional losses exceeded the theoretical calculation. Moreover, to address the threading issue of the 850 ft (260 m) tendons, the ratio of duct area to the area of strands in a tendon was specified as a minimum of three. Figure 10 presents Stage 1 and Stage 2 post-tensioning details, while Fig. 11 shows the reinforcement details including all the prestressing used at different phases of construction.
CONSTRUCTION

Construction of the bridge’s substructure and erection of the girders took two in-water construction windows to be completed. If temporary supports had been required for casting of the intermediate splices, another construction window would have been necessary, extending bridge completion for another year.

The contractor was responsible for worker safety during construction. Indeed, the scheme for the transportation of the girders and their subsequent erection was technically sound and appropriate for the long girders and the road conditions at the construction site.

Accidental torsional buckling of the girders was avoided by providing a pinned front dolly and a torsionally stiff, back double dolly. The use of a rear tandem dolly system, with a rotational stiffness of approximately 260,000 k-in./rad (29,000 kN-m/rad), far exceeded the design requirement. Figure 12 shows the transportation configuration.

It should be noted that the equipment used to transport the spliced girders is intended for moving heavy loads over a short distance; it is not appropriate for use on the open road and would not meet the permitting requirements for such a haul. The ability to use such equipment was one advantage of assembling the girders at a staging yard near the site.

The lateral stability of the girders was sensitive to the distance from the end of the girder to the pick point (distance a). A relatively small increase in a increases the girder’s lateral stability considerably. In the contract drawings, the distance a was specified as 15 ft (4.6 m). The contractor proposed reducing this distance to only 7 ft (2.1 m) to accommodate the capacities of the cranes and the work bridge.

Despite a marginal safety factor, which was slightly higher than the design (or code-required) minimum, all girders were lifted and placed on top of the piers satisfactorily. Figure 13 shows the erection of these 177.5 ft (54 m), 122 ton (1085 kN), spliced W95PTG girders.

Because of the large diameter ducts within the girder webs, there was inad-
Fig. 10. Stage 1 and 2 post-tensioning details.

Fig. 11. Spliced girder reinforcing details.
equate space on either side of the duct and on the faces of the girder webs to embed high-strength tensile bars or strands to serve as pick-up hoops. Consequently, an external lifting assembly was developed.

This assembly included four high-strength tensile bars, two on either side of the girder web, which were post-tensioned at the site prior to transportation and subsequent lifting. The external lifting concept was included in the bridge’s contract drawings. The contractor designed, supplied, and used the lifting devices successfully.

The contractor used a temporary work bridge throughout the construction of the bridge’s substructure and during girder erection. It was installed in the river channel, and was later removed at the beginning and end of each of the 2001 and 2002 construction windows. The work bridge comprised a timber deck, steel girders, and steel pipe piles.

Drilled shafts were installed by a Leffer oscillator operating from the work bridge’s finger extensions. Two difficulties were encountered initially. Due to the large frictional forces that were generated between the interior face of the drilling casing and the partially hardened concrete, large extracting forces were created during the removal of the casing. These forces were initially counteracted by four reaction piles per shaft platform, but as the bearing capacity of the bearing piles was exceeded by the platform supporting the oscillator operation, the whole platform tilted. This problem was resolved by increasing the penetration depth of the platform’s reaction piles and by increasing the number of piles from four to eight per platform.

The other difficulty consisted of some casing extraction problems due to the hardening of the concrete. Concreting the shaft in predetermined increments of 20 ft (6 m) and extracting the shaft’s casing segments during the reverse-oscillation activity resolved this difficulty.

This procedure continued until concrete was placed in the entire shaft and the shaft casing was completely extracted. Keeping the time it took to unbolts and remove a given casing segment to under an hour was the key to a successful operation, as well as using concrete with retarding admixtures to delay hardening.

Due to the complex nature of the spliced girder deflections and the variability of the material properties among the girder segments, prediction of deflections—even with a nonlinear time-dependent analysis—was expected to be imprecise. To prevent the possibility of having a sagging bridge, an adjustment was introduced to the contract drawings during the design phase. This adjustment was made possible by introducing a variable haunch height on top of a given girder.

The variable haunch height was defined as the $x$ distance in the contract drawings. Prior to placing the deck, elevations of the top of the girders were

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Fig. 12. Spliced girder hauling activities with pinned front dolly and torsionally stiff back double dolly.

Fig. 13. Spliced girder erection activities showing external lifting assembly.
determined and the values of the \( x \) distances were calculated for each girder. Hence, the desired grade of the bridge was obtained both longitudinally and transversely. Figures 14 and 15 show the construction of the bridge at various stages of creation.

**CONCLUSION**

In this article, the authors have discussed special design and construction challenges, and their associated engineering solutions, in the construction of the Old 99 (Riverside) Bridge. Its successful completion demonstrates that spliced-girder technology can be applied to girder lengths that are beyond the customary spans and transverse spacing of standard precast, prestressed concrete girders. This particular method of bridge construction requires no precast pier segments or temporary, intermediate supports.

As a future enhancement, and to fully mobilize the potential of this concept, it is recommended that the American Association of Highway Transportation Officials load-resistant factor design method of calculating live load and impact moments be used, especially where the entire deck is analyzed as a grid, including any transverse post tensioning of the intermediate diaphragms.

In 2004, the Old 99 Bridge won a PCI Design Award for “Best Bridge with Spans Greater than 135 ft.” The jury comments were as follows:

“We think this type of bridge will be used more and more often in the future. It uses girder segments spliced together at the job site to eliminate transportation and site constraints. Assembling the girders near the site and erecting them in one piece eliminates falsework problems. This technique enabled 180-ft spans; it also helps get piers out of the way, which in this particular Northwestern environment, were a problem in collecting debris. If these girders were cast in one place, they would have weighed 122 tons, which would have precluded shipping them to the site. So, splicing them at the site made the project feasible by also eliminating the need for falsework in the river.”

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