INNOVATIVE USE OF SELF CONSOLIDATED CONCRETE (SSC) FOR PRECAST CONCRETE GIRDERS

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ABSTRACT

Concrete has been an integral part of the history and advancement of bridge design and construction. Modern advancements in technology have created concrete bridges that have lower construction costs and better long term characteristics than bridges made primarily of steel. One of the modern technological advancements that improves the quality and constructability of concrete bridges is Self-Consolidating Concrete (SCC). This technological advancement was recently demonstrated in year 2015 with the replacement of the Potter Road Bridge that crosses the South Fork Nooksack River in Whatcom County, Washington. The project utilized SCC to construct the superstructure girders of the new bridge that replaced the old structurally and functionally deficient steel thru truss bridge.

Various project constraints that include profile, alignment, clearance envelope's and high seismic demands all contributed to the use of shallow precast prestressed and precast post-tension spliced girders that were needed to span the entire river with a 240-foot long main span. The use of SCC greatly improved the quality and constructability of the superstructure girders. This was one of the first uses of SCC for precast superstructure concrete girders in the state of Washington. The paper will discuss in detail on how the project succeeded with the use of SCC.

Keywords: Self Consolidated Concrete, Precast, Post-Tensioned, Bridge Replacement, Seismic Design

INTRODUCTION

PROJECT HISTORY

The original Potter Road Bridge over the South Fork Nooksack River in Whatcom County, Washington was constructed in 1927. It consisted of 7 short timber approach spans on the west side of the river, a main river span made up of a single 150 foot long steel thrrough truss, and 9 short timber approach spans on the east side of the river. Over the years the timber approach spans proved problematic as they regularly caught debris during high flow events and restricted the hydraulic opening at the crossing. Ultimately in 1974, the timber approach spans were replaced with 31 foot long concrete channel beam approach spans, and a 76 foot long overflow structure was constructed roughly 450 feet to the southwest of the main river crossing. Refer to Figure 1 for an overview of the old bridge.

Fig. 1 Looking Northwest at the Old Bridge

The narrow two lane bridge does not carry high volumes of traffic; however, it is very important to the local rural community as the detour route is significant at almost 50 miles. Over the years the river hydraulics at the site had changed. During seasonal high flow events the west end of the bridge was regularly overtopped and this significantly impacted the local community. The bridge was already considered functionally and structurally deficient when the original Type, Size and Location (TS&L) Report was issued in year 2003. Replacement was considered the preferred option for the new bridge. However, the project had to wait an additional 8 years after the original study until appropriate funding was available to complete the replacement bridge design and construction.

PROJECT CONSTRAINTS

There are a number of factors that make this a particularly challenging project. The first major challenge was project funding. The rural county had very little funding available themselves for capital transportation improvement projects. As such, funding from other sources was needed to be able to fund a substantial portion project. In the state of Washington, the Washington State Department of Transportation (WSDOT) developed the Bridge Replacement Advisory Committee (BRAC) funding program to help local agencies fund capital transportation improvement projects specifically related to bridge rehabilitation and bridge replacement projects. The county competed for funds in 2004 and 2006 and was unsuccessful. Then in 2008 the funding was suspended due to the national economic recession. Finally in 2010 the county was successful to getting final design and construction funding for the project.

The project site also had significant alignment and stage construction challenges. The vertical roadway profile of the new bridge had to be virtually the same as the existing bridge because the county did not want to also replace the nearby overflow structure. This proved to be challenging as the existing 1927 vertical profile did not meet current roadway drainage standards. Additionally, the hydraulic opening of the bridge needed to be increased without significantly raising the existing vertical profile. Finally, in addition to the geometric constraints, the detour length precluded closing the existing bridge during construction. Subsequently a very cost effective way of replacing the bridge needed to be determined during the design phase for the project.

LONG SPAN BRIDGE REPLACEMENT STRUCTURE

With the challenging project constraints in mind, a unique solution was needed to replace the aging bridge on this locally important route. The TS&L Report studied a number of alternatives that considered improving the hydraulic opening, maintaining a vertical alignment as close to the original alignment as possible and stage construction sequences that would maintain use of Potter Road during construction. Ultimately a two span configuration was chosen to be the preferred option.

The new bridge has a 28 foot wide roadway that supports two 11 foot wide lanes. The main river span is a 240 foot long span that is made up of four WSDOT WF95PTG (95” deep) prestressed post-tensioned spliced concrete girders that are spaced at 7.5 feet on-center (O.C.). The east approach span consists of a 120 foot long span that is made up of four WSDOT WF50G (50” deep) prestressed concrete girders that are spaced at 7.5 feet O.C. Refer to Figure 2 for typical superstructure cross sections of the new bridge. The new bridge is constructed on an alignment that is located just south of the existing bridge alignment. This span configuration maximized the width of the hydraulic opening and the two superstructure types maximized the height of the hydraulic opening while still maintaining a vertical profile that was very similar to the existing vertical profile.

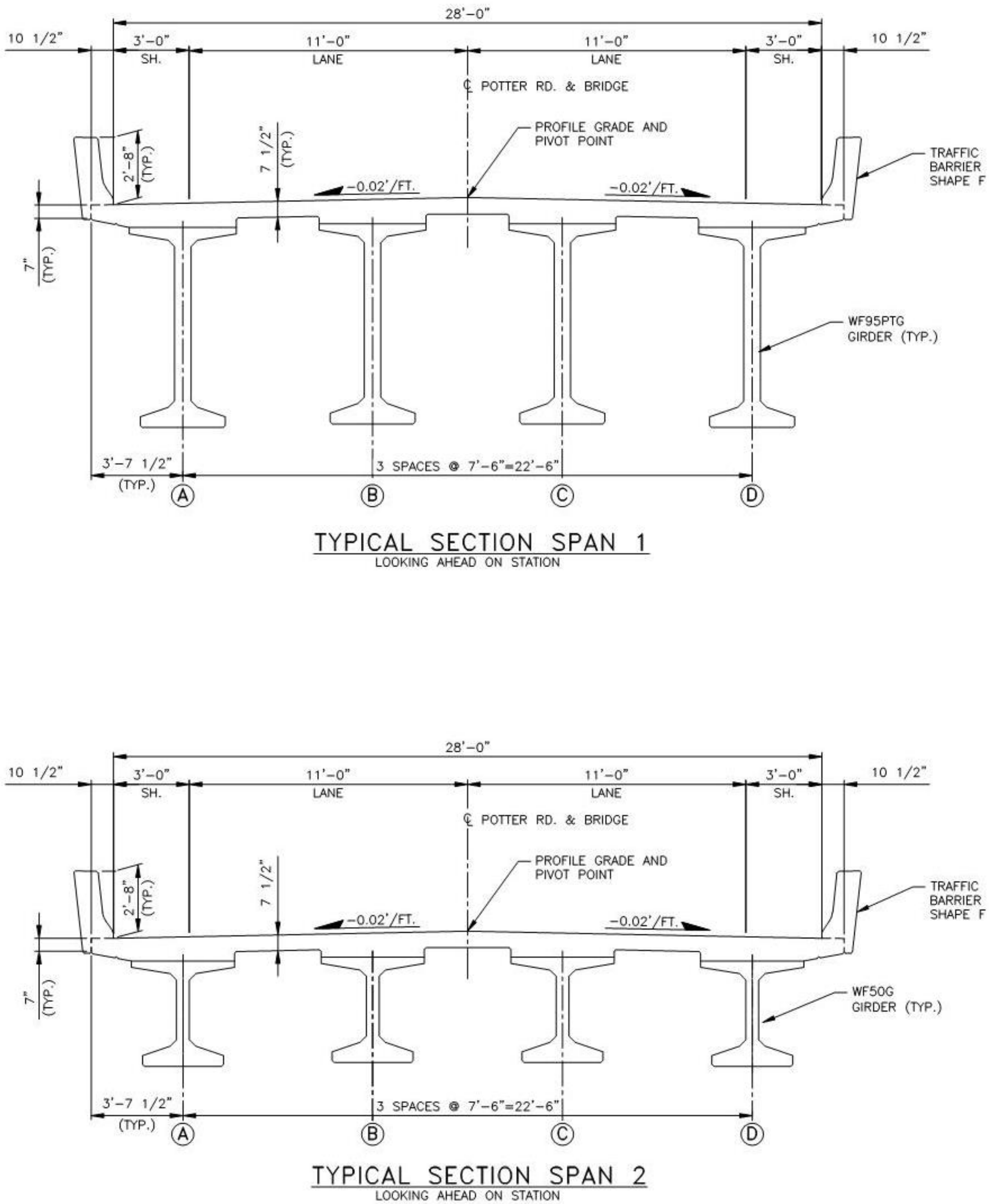

Fig. 2 Span 1 River Crossing and Span 2 East Approach Typical Sections

Only a single 5 foot diameter column was needed for the center pier to support the bridge, which also minimizes the possibility of debris accumulation during high flow events.

DESIGN CRITERIA

BASIC DESIGN REQUIREMENTS

Long span structures commonly require additional project specific design criteria, and this project is not different. The WSDOT Bridge Design Manual, along with the AASHTO Bridge and Seismic Design Specifications, set the basis for the project design requirements. Additionally the unique post-tensioning aspects of the projects required the adoption of PTI Post-Tensioning Manual in order to adequacy design the replacement structure.

SEISMIC DESIGN REQUIREMENTS

A number of project constraints made the seismic design of this long span structure challenging. The Peak Ground Acceleration (PGA) for the project site was determined to be 0.286, and this combined with an F Site Class put the bridge in a Seismic Performance Zone of 4. This required the foundation elements that consist of 2.5 foot diameter concrete piles to be capacity protected and the center pier had to satisfy plastic hinging requirements.

Additionally, there was a strong desire to minimize the size of the center pier cap so as to maximize the size of the hydraulic opening during high water events. Since the two spans are two different superstructure types, we could not make them continuous over the center pier. WSDOT BDM doesn't permit the use of multiple simply-supported spans, Earthquake Resisting System (ERS) number 6. With WSDOT approval, the design team was able to use longitudinal restrainers that connect the WF95PTG prestressed post-tensioned spliced concrete girders to the WF50G prestressed post-tensioned concrete girders.

HIGH PERFORMANCE SUPERSTRUCTRE CONCRETE

The project team needed to limit the depth of both superstructure spans in order to maximize the size of the hydraulic opening. This is primarily why there are two different superstructure types for the bridge between Spans 1 and 2. However, in order to achieve the most economical solution the project design team needed to evaluate the ability to obtain high performance concrete (HPC) in the northwest region of the United States. It was determined that high performance concrete with a 28 day compressive stress in excess of 10.0 ksi was readily available in the region. However with the premium costs associated with the 10.0 ksi plus high strength concrete mix the design team decided to adopt a 28 day 9.0 ksi compressive stress concrete with 6ksi release strength to be used for the WF95PTG girders and a 28 day 8.5 ksi compressive strength concrete with 7ksi release strength to be used for the WF50G girders. This selection proved to fit well between the design requirements and funding capabilities of the project.

SUPERSTRUCTURE DESIGN

POST-TENSIONED SPLICED GIRDER DESIGN

The WSDOT Bridge Design Manual (BDM) provides sound guidance to designer's who are faced with the design of long span post-tensioned spliced concrete girders. A variety of design considerations in the WSDOT BDM are as follows;

- Temporary concrete stresses shall be checked at each stage of pretensioning and post-tensioning, considering all applicable loads during construction.
- Consider bracing girder segments after erection and before post-tensioning.
- Post-tensioning may be applied before and/or after deck concrete placement.
- Sequence of placing concrete for the closure joints and deck shall be specified.
- The length of closure joints shall not be less than 2 feet.

The design of the WSDOT WF95PTG prestressed post-tensioned spliced concrete girders had a number of unique challenges that needed to be overcome. The spliced girders needed to be transported to the site in precast segment lengths no greater than 120 feet. This was because the roadway curves along the haul route could not support precast segments lengths any greater than 120 feet. As a result of this restriction, post-tensioned spliced girder segments measured 58 feet for Segment 1, 118 feet for Segment 2 and 59 feet for Segment 3. Final service stresses were limited by a zero tension limit and a compression limit of up to 0.6*f'c in both the precast girders and cast-in-place closure joints. Refer to Figure 3 for Segment 1 under construction at the precast facility.

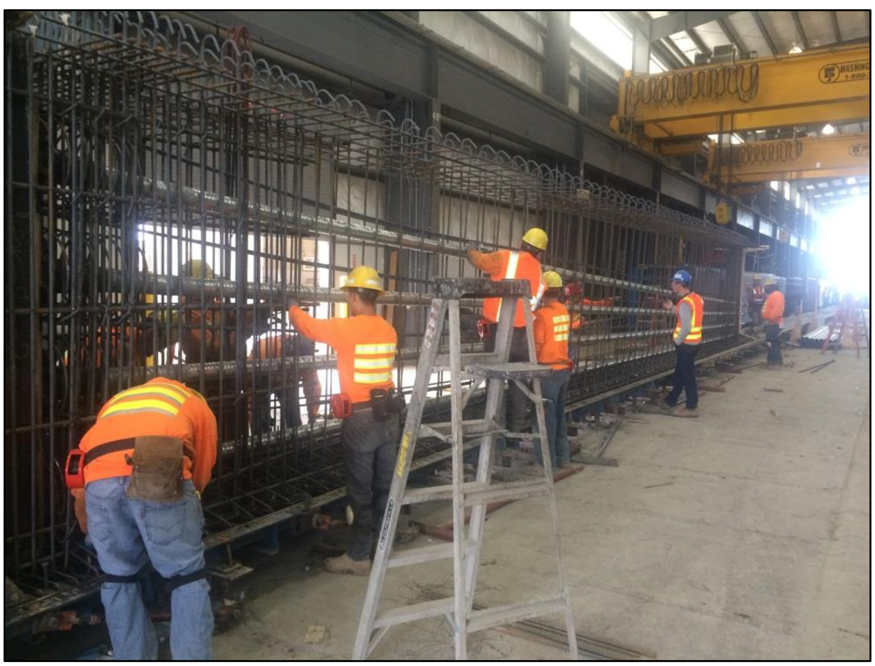

Fig. 3 Post-Tensioned Spliced Girder Segment 1 Girder Segment Construction

To meet transport, temporary and erection stress limits, Segments 1 and 3 utilized 4 prestressing strands in the bottom flange, and Segments 2 utilized 16 prestressing strands in the bottom flange. These strands consisted of 0.6 inch diameter ASTM A416, Grade 270, low relaxation prestressing strands. The girders were permitted to be released from the forms when the concrete reached a release strength of 6.0 ksi. Shear reinforcement consisted of #5 Grade 60 steel reinforcing bars that were spaced as close as 3 inches O.C. at the ends of each segment and up to 12 inches O.C. at the center of each segment. Refer to Figure 4 for Segments 3 being stored at the precaster's facility.

Fig. 4 Stored Post-Tensioned Spliced Girder Segment 3 Girder Segment

To meet final service stress limits, 4 full length 22 strand post-tensioned tendons utilizing 0.6 inch diameter ASTM A416, Grade 270, low relaxation prestressing strands are installed in each WSDOT WF95PTG girder at the completion of span deck construction.

PRESTRESSED CONCRETE GIRDER DESIGN

The design of the WSDOT WF50G prestressed concrete girders generally followed standard WSDOT BDM practices. A total of 46 strands (12 harped strands and 34 straight strands) were required in each girder to meet the necessary service and strength design requirements. The girders were permitted to be released from the forms when the concrete reached a release strength of 7.0 ksi. Like the prestressed post-tensioned spliced concrete girders, 0.6 inch diameter ASTM A416, Grade 270, low relaxation prestressing strands were used. Shear reinforcement consisted of #5 Grade 60 steel reinforcing bars that were spaced as close as 3 inches O.C. at the girder ends and up to 12 inches O.C. at midspan.

CONSTRUCTION SPECIFICATIONS

SUPERSTRUCTURE ERECTION PLAN

The project site required specific restrictions to the superstructure erection plan that needed to be included in the original bid documents. In order to satisfy environmental requirements, the locations of the temporary bents and performance requirements for the temporary work access bridge for the prestressed post-tensioned spliced concrete girder span had to be prescribed in the design drawings. The specific required elements for the temporary bents and temporary work access bridge into the river were defined in the project special provisions as follows;

- A diversion system to divert stream flow to the center of the channel had to be a minimum of 60 feet wide to permit fish passage through the work zone.
- Sheet pile cofferdams need to meet WSDOT Standard Specifications, Section 2-09.3(3)D, and were only permitted to be vibrated into the ground.
- Fish exclusion of the rock jetties needed to be provided per the WSDOT fish exclusion protocol in the special provision appendices.
- Dewatering of any rock jetty cofferdams needed to be discharged to an upland area that was isolated from the river.
- A geotextile mat needed to be installed on the riverbed inside the rock jetty cofferdam prior to installing the riprap jetty.
- The temporary work access bridge abutments could be installed within the rock jetty cofferdams utilizing steel H-piles that could be installed with a crane-mounted impact pile driver “in the dry”, through the riprap fill.

The contractor was limited to an in-water work period between July 15 and October 15 where all in-water work needed to be completed over two consecutive seasons. Ultimately the contractor was able to develop an effective temporary bent system that only utilized steel pipe piles and did not require the use of a temporary cofferdam system. Additionally, the contractor built the temporary work access bridge using a system of steel pipe piles and crane mats. This was beneficial to the project as it impacted the river to a lesser degree than the limits described in the project special provisions.

The WSDOT Standard Specifications are also well adapted to prestressed post-tensioned spliced concrete girder and prestressed concrete girder construction. The agency has been a national leader in the development of prestressed concrete design and construction for a number of decades. This long term knowledge by both the state and local precast industry aids general contractors in the development of innovative and cost effective girder erection methods for the individual project sites they work on. Refer to Figure 5 showing Segment 3 of the prestressed post-tensioned spliced girder span being erection into its final position.

Fig. 5 On-Site Erection of Post-Tensioned Spliced Girder Segment 3 Girder Segment

HIGH PERFORMANCE SUPERSTRUCTURE CONCRETE

The WSDOT Standard Specifications provides a significant amount of flexibility to precasters in the development and implementation of their HPC mix designs primarily due to the local industries long history in developing precast bridge elements (that dates back to the early 1950's) and long working relationship with WSDOT. Consequently when innovations are presented by local precasters, WSDOT and associated local agencies are very open to working with the precasters to develop the details and ultimately advance the precast concrete industry. This willingness of WSDOT and associated local agencies was critical to this project being able to implement high performance SCC in the precast superstructure elements.

SUPERSTRUCTURE CONSTRUCTION

POST-TENSIONED SPLICED GIRDER CASTING USING TRADITIONAL HPC

Casting of the WSDOT WF95PTG prestressed post-tensioned spliced concrete girders had a few challenges, but ultimately worked out well. The WF95PTG is a newer cross section that had been developed by WSDOT and local precasters last decade to be able to accommodate spans well over 200 feet. The precaster had cast WF95PTG girders in the past; however, the frequency of use of these girders is much less than the WSDOT WF83PTG girders. As such the precaster requested during the early stages of construction to substitute the WF95PTG girders with WF83PTG girders as their operations for the shallower girders is much more efficient. Unfortunately the request was not able to be granted as the WF83PTG girders could not achieve the necessary span length of 240 feet for Span 1. The precaster continued with the original design and successfully cast the WF95PTG girders at their facility located at the Port of Tacoma in Washington. Refer to Figure 6 showing Segment 2 in their casting facility just after the forms were stripped.

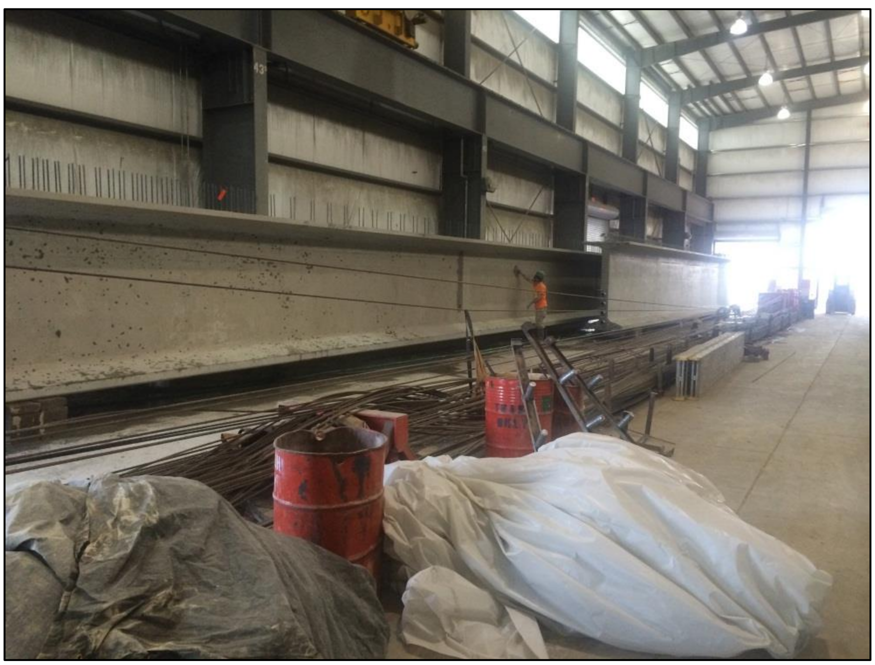

Fig. 6 Casting of Segment 2 Girder Segments

A significant challenge experienced by the precaster involved proper consolidation of concrete in the very heavily reinforced precast post-tensioned concrete girders. The deep and narrow girder section, combined with the very closely spaced shear reinforcing steel, post-tensioning ducts and post-tensioning anchorages, made proper concrete placement difficult particularly at the ends of the girders. Even with the high levels of congestion in the girder cross section, placement of concrete still went well. However, a fair amount of advanced planning was required to setup the proper concrete placement procedure.

PRESTRESSED GIRDER CASTING USING HIGH PERFORMANCE SCC

Span 1 girder segments were cast before Span 2 girders. As the precaster was preparing their shop drawing submittals for Span 2, they considered the casting of the WSDOT WF50G prestressed concrete girders to be a good opportunity to use high performance SCC for precast girder construction. Note that the use of SCC for WSDOT and local agency transportation projects is not new in the state of Washington; however, its use in primary bridge structural members was only recently permitted in 2014 in the WSDOT BDM. Prior to 2014, the WSDOT BDM only permitted the use of SCC in the construction of precast noise wall panels, barriers and other non-primary structural members.

Since early 2014, there had been three projects prior to the Potter Road project that utilized SCC for primary bridge structural members in Washington. The precaster had been involved in two of those prior projects, and had been using SCC in the state of Washington for the permitted non-primary structural elements indicated previously. From this experience they were able to successfully develop and implement a high performance SCC mix design for the precast prestressed concrete girders that met the project testing requirements indicated in Table 1.

Testing Requirements for HPC		
Property	**Test Method**	**Acceptance Criteria**
Slump Flow	ASTM C 1611	+/- 2 inches
Visual Stability Index (VSI)	ASTM C 1611, Filling Procedure B	1 max
T50 Flow Rate	ASTM C 1611, Filling Procedure B	Less than 6 seconds
Static Segregation	ASTM C 1610	10 % max
Hardened Visual Stability Index (HVSI)	AASHTO PP58	1 max
J Ring Passing Ability	ASTM C 1621	1.5 inch max
Air Content	WSDOT T 818	4.5% < AC < 7.5%
Unit Weight	AASHTO T 121	For Information Only
Temperature	AASHTO T 309	50ºF < T < 90ºF
Elasticity (E=Modulus of Elasticity)	ASTM C469	For Information Only
Compressive Strength (at 28 days)	AASHTO T22	>8,500 psi (all tests)

Table 1 High Performance SCC Concrete Testing Requirements

Compressive strength was not a major concern for WSDOT and the owner as the high performance SCC mix design for the WSDOT WF50G prestressed concrete girders achieved an average compressive strength of 11,280 psi, as calculated per ACI 318, Table 5.3.2.2. During the pre-approval process the main points of concern that needed to be addressed where related to proper distribution of aggregate and segregation of the high performance SCC mix design. These two items had been problematic for WSDOT and other local agencies in past projects, so the trial mix needed to properly address these two items in addition to the other previously mentioned testing requirements.

The high performance SCC mix design slump test starts like a standard slump test, although many testing technicians will turn the cone upside down to make it easier to fill. When the cone is lifted, the SCC spreads out like pancake batter. The slump flow is measured as the diameter of the pancake. Our SCC mix was able to successfully achieve a Visual Stability Index of 0 with a flow rate of 5 seconds and a static segregation of 7.1%. Additionally it was able to achieve a J Ring Passing Ability of 1 inch and maintain an HVSI of 0. The SCC mix design also successfully passed all remaining concrete testing requirements indicated earlier. WSDOT and the local agency owner were very satisfied with the High Performance SCC mix design developed by the precaster. Refer to Figure 7 showing resulting Slump Flow test with J ring still in place.

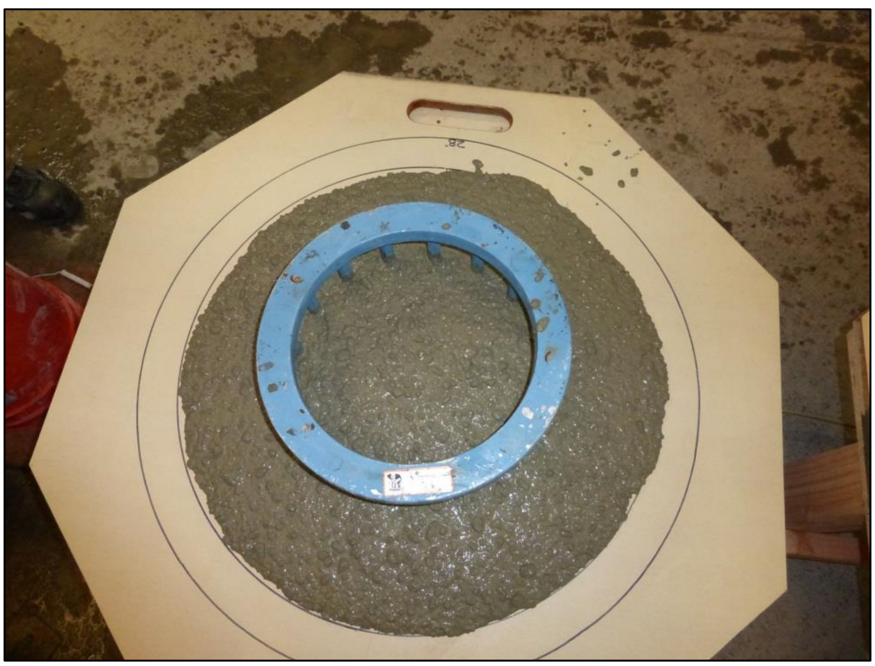

Fig. 7 Slump Flow Testing of High Performance SCC Mix Design

Casting of the WSDOT WF50G prestressed concrete girders proceeded shortly after the approval of the high performance SCC mix design. The precaster's previous experience with SCC was essential in their ability to deliver a high quality product with such a short production run (four 120 foot long girders total). The precaster's batch plant was quickly modified to accommodate mixing of the high performance SCC mix. Their delivery system from the batch plant to the forms was already well adapted to transporting the SCC mix.

Most importantly was the precaster's staff experience in placing high performance SCC for precast elements. A feature of SCC is that vibration of the placed concrete is not required as the mix is able to flow easily through highly congested regions in a form. The precaster found on previous projects using SCC that in order to produce very high quality precast elements, a small amount of vibration is needed to obtain a very clean finish. Refer to Figure 8 showing placement of high performance SCC in the WF50G prestressed concrete girders forms. Note the straight vertical girder bars.

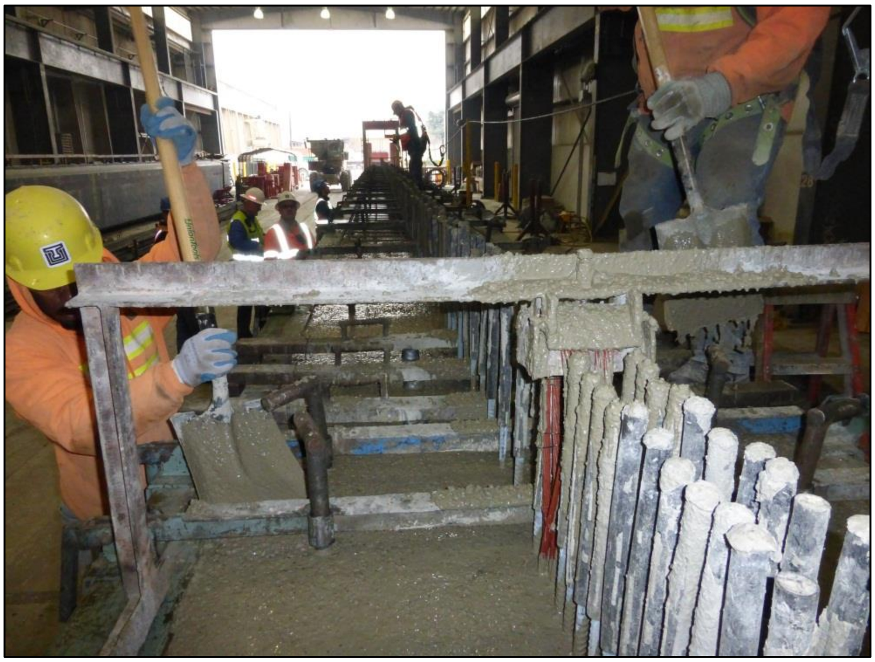

Fig. 8 Placement of High Performance SCC in Span 2 Girder Forms

Production casting of the WF50G prestressed concrete girders was completed on schedule, and WSDOT and the local agency owner were very satisfied with the production of the prestressed girders.

SUPERSTRUCTURE ERECTION

With the project restrictions previously mentioned, a complex girder erection plan needed to be developed by the contractor to erect the three girder segments of each WSDOT WF95PTG prestressed post-tensioned spliced concrete girder that make up the 240 foot long Span 1. Each Segment 1 weighed approximately 90 kips, each Segment 2 weighed approximately 164 kips, and each Segment 3 weighed approximately 90 kips. The weights of the girders combined with the temporary construction access restrictions, and desire to use cost effective wheel based cranes, were the primary factors in developing and executing the project specific superstructure erection plan.

The contractor decided to erect all for Segment 2 girder segments first, then erect all Segment 1 girder segments, and finally erect all Segment 3 girder segments. Erection of the center Segment 2 was challenging primarily due to the limitations imposed on the wheel based cranes at the project site. A single wheel based crane could not be used to erection the center Segment 2 girder segments, and a single crawler crane with the necessary capacity was cost prohibitive to the project. The contractor was able to obtain a Grove GMK 7550 550 ton crane and a Grove GMK 6300B 300 ton crane for the erection of Span 1 girder segments. The Grove GMK 7550 was positioned on the west shoreline as it could obtain the greatest reach necessary for the project of 148 foot boom with a capacity of 88 kips. The GMK 6300B was positioned on the end of the eastern temporary work bridge where it could still provide a capacity of 88 kips at a 108 foot boom.

For Segment 2 girder segments, a method was developed where one end of the girder segment was picked up off the transport truck by the Grove GMK 6300B. With the other end of Segment 2 girder segment still on the transport truck, the truck began backing up to the west while the Grove GMK 6300B boomed out simultaneously. Then over the river, the Grove GMK 7550 boom towards the girder segment end and connected to the triangular transfer beam that the Grove GMK 6300B was connected to. The load of the west end of Segment 2 girder segment was then fully transferred to the Grove GMK 7550. Refer to Figure 9 showing the transfer of the west end of Segment 2 girder segment between the two cranes over the river.

Fig. 9 Transfer of the West End of Segment 2 Girder Segment

The Grove GMK 6300B disconnected from the west end of the girder and then connected to the east end of the girder still on the transport truck. Both cranes (each supporting one end of the segment) then lifted, positioned and placed Segment 2 girder segment on the temporary bents. Once all four Segment 2 girder segments were placed, Segment 1 girder segments also needed to utilize a unique erection sequence. The Grove GMK 6300B first picked the Segment 1 girder segment off of the transport truck on the east temporary work bridge and then positioned it over the erected Segment 2 girder segments. Note that prior to lifting the first Segment 1 girder segment, the Segment 2 girder segments were braced together and a bearing frame was constructed on top of the girder segments. Segment 1 girder segment was then set on top of the erected Segment 2 girder segments. Then the Grove GMK 6300B connected to the east end of Segment 1 girder segment and the Grove GMK 7550 connected to the west end of Segment 1 girder segment. Both cranes lifted the segment and shifted it further to the west and set the segment back down on the erected Segment 2 girder segments. Both cranes disconnected and the Grove GMK 7550 reconnected to Segment 1 girder segment, picked the segment and placed it in its final position on the west abutment and temporary bent.

After erection of the Segment 1 girder segments was complete (the other three Segment 1 girder segments followed the same sequence previously described) then Segment 3 girder segments were erected. The erection of the Segment 3 girder segments was relatively simple where the Grove GMK 6300B was used to pick and place the girders segments from the east temporary access work bridge onto the other temporary bent and Pier 2.

Span 1 intermediate girder diaphragms were then cast to connected all girder segments together. The concrete deck for Span 1 was then cast, and when it reached design strength the WSDOT WF95PTG prestressed post-tensioned spliced concrete girders were post tensioned together. Refer to Figure 10 showing the post-tensioning of the girder segments.

Fig. 10 Post Tensioning of Girder Segments at Abutment 1

The deck needed to be cast before post-tensioning of the girder segments in order to achieve the design span length of 240 feet. Once post-tensioning operations were complete, the end diaphragms were cast for Span 1 and the remainder of the Span 2 superstructure was erected.

CONCLUSION

The project will be completed in late 2015, with most of the bridge construction already now complete. In general the project has been a very successful project with the unique achievements of a 240 foot long prestressed post-tensioned spliced girder simple span and another successful implementation of high performance SCC in primary superstructure elements in the state of Washington. Refer to Figure 10 showing Span 1 during construction.

Fig. 11 View of New 240 foot long Span 1 looking Northwest

This project has created a unique long span bridge for Whatcom County. Additionally, the successful implementation of high performance SCC on this project finalized the necessary successful project support needed by WSDOT to now be able to fully permit the use of high performance SCC in primary superstructure members. Language has been drafted for the WSDOT Standard Specifications and is currently under agency review. WSDOT is planning to fully implement the use of SCC in their Standard Specifications in the 2016 edition of the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction.

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