New Deep WSDOT Girders Used for the Twisp River Bridge

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This article provides details of the design and construction of the Twisp River Bridge, a 60.0 m (197 ft) long single-span bridge in the North Cascade Mountains in Washington State. New 2.4 m (7.87 ft) deep WSDOT W95PTMG girder sections were used in the construction. The girders were delivered to the site in three precast, pretensioned segments, erected on falsework bents, and post-tensioned together after the roadway deck was placed. High performance concrete (HPC) with a 28-day strength of 55 MPa (8.0 ksi) was used for the girders. The girder closure pours required a strength of 35 MPa (5.0 ksi), while all other cast-in-place concrete had a specified strength of 28 MPa (4.0 ksi). High-load reinforced elastomeric bearings, designed by AASHTO Method B, were used at the ends of the girders. This was the first use of elastomeric bearings by WSDOT in a post-tensioned bridge. All post-tensioning anchorages were placed in a cast-in-place concrete end diaphragm. The authors provide a summary of the design features of this project, the challenges involved, and the lessons learned from it.

The new Twisp River Bridge is a spliced single-span 60.0 m (197 ft) semi-integral precast, prestressed concrete structure (see Figs. 1 and 2). The bridge is located in the City of Twisp in Okanogan County, midway between Seattle and Spokane on State Route 20, which is the only highway that passes through the rugged North Cascade Mountains in Washington State. The Washington State Department of Transportation (WSDOT) is the owner and designer of the new bridge.

Fig. 1. Panoramic view of the Twisp River Bridge, Okanagon County, Washington State (picture taken in February 2002). Courtesy: Alex Young, WSDOT Bridge Architect.

A plan and elevation of the Twisp River Bridge is shown in Fig. 3. The structure replaces a four-span cast-in-place concrete T-beam bridge, built in 1935, that had become functionally obsolete (see Fig. 4). The bridge crosses the Twisp River, which flows into the Methow River.

The Twisp and Methow rivers are home to several endangered fish species: the Upper Columbia Steelhead, the Upper Columbia Chinook Salmon, and the Bull Trout. Under normal circumstances, WSDOT would have designed a two-span prestressed concrete girder superstructure with an intermediate pier in the river. Environmental restrictions, however, allowed work below the ordinary high water line only during the months of July and August, which make the installation of an intermediate pier difficult.

This obstacle meant that WSDOT would have to come up with a new innovative solution. Prior to the availability of the W95PTMG sections, the likely scheme would have been to span the river with steel plate girders. But in this case, the solution was a single-span bridge using the new deep WSDOT girder sections, which would eliminate construction in the river during the September to June fish closure.

WSDOT received $500,000 of supplementary funding from the Federal Highway Administration's (FHWA) Innovative Bridge Research and Construction Program for the new design features and construction techniques used to advance the state of practice in transportation structures.

BACKGROUND

Currently, there are roughly 3000 bridges on the Washington State highway system. Approximately 87 percent of these bridges are concrete structures, about half of which are prestressed. Today, WSDOT uses precast, pretensioned concrete girders with cast-in-place decking as its standard concrete bridge.

For many years, the deepest standard girder was the W74MG, which is 1867 mm (73.5 in.) deep and weighs 12.1 kN/m (0.83 kips per ft). These girders are typically used for bridges in the 36.6 to 42.7 m (120 to 140 ft) span range, although spans of up to 48.8 m (160 ft) are possible.

Fig. 2. Closeup of bridge showing new 2.4 m (7.87 ft) deep, W95PTMG girder.

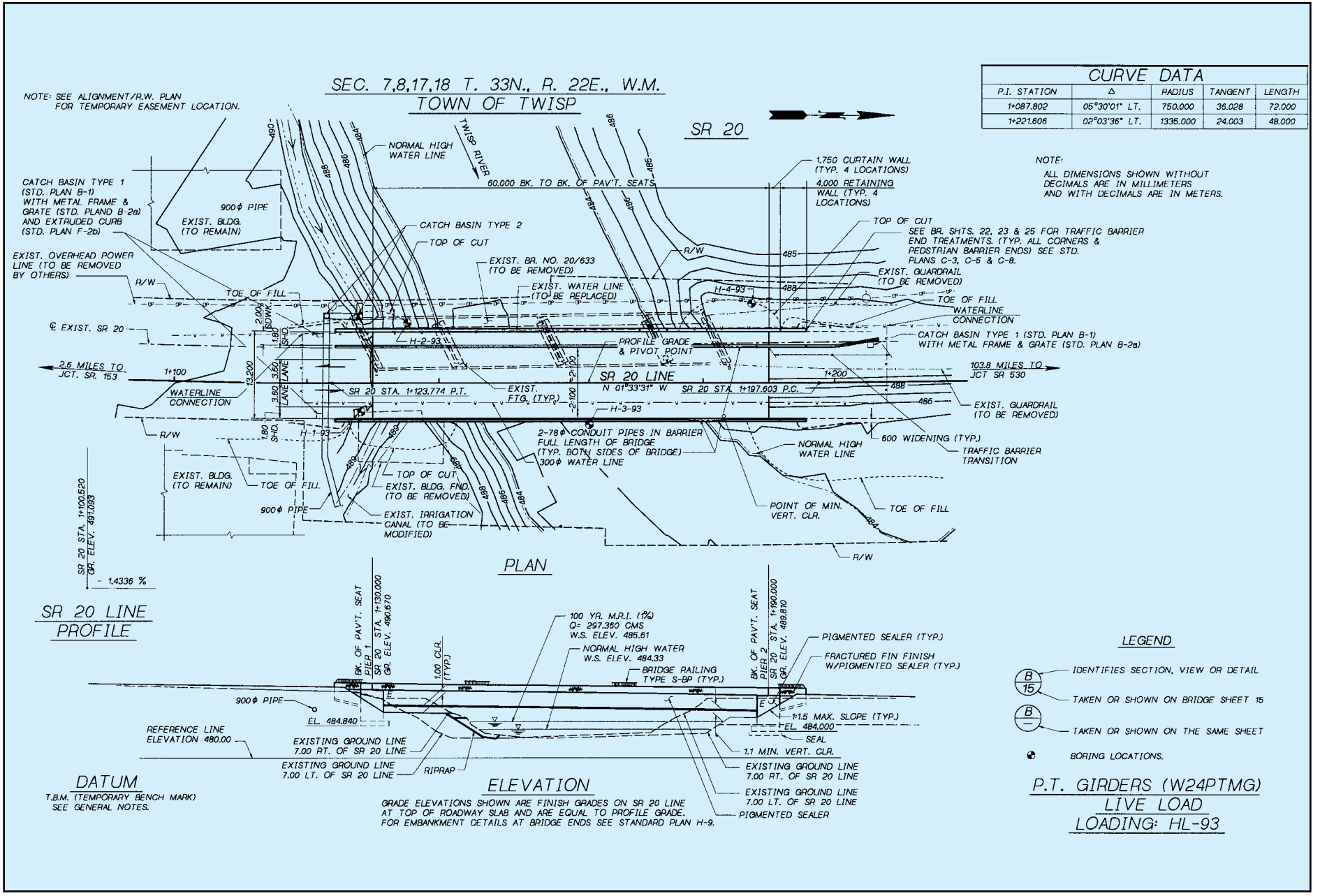

Fig. 3. Plan and elevation of Twisp River Bridge replacement structure.

Fig. 4. Obsolete cast-in-place reinforced concrete T-beam bridge.

At their 1996 annual meeting, the Pacific Northwest Precast/Prestressed Concrete Institute (PNW/PCI), a joint industry and WSDOT team, decided to develop new deeper WSDOT standard prestressed girder sections. The precast/prestressed concrete industry took the lead in this effort, which resulted in two new pretensioned sections.[1] These are the W83MG, which is 2100 mm deep and weighs 16.2 kN/m (82.68 in. and 1.114 kips/ft), and the W95MG, which is 2400 mm deep and weighs 17.5 kN/m (94.49 in. and 1.196 kips/ft).*

Because of the great efficiency of these sections, a third section, the WF74MG, which is 1850 mm deep and weighs 15.3 kN/m (72.83 in. and 1.045 kips/ft), was subsequently added as a standard section. These girders supplement the W74MG section and are shown in Fig. 5.

Span lengths for the two deeper pretensioned sections are limited primarily by the ability of precast plants and hauling trucks to handle the girder weight. Fabrication plant handling capacity is limited to 890 kN (200 kips), and trucking equipment readily available in Washington State can accommodate loads of up to 810 kN (182 kips). The maximum legal load limit in Washington is 890 kN (200 kips), beyond which a special permit is required. These restrictions control the maximum span that can be handled in the fabrication plant and shipped to the bridge site.

Based on the 890 kN (200 kips) limit, the maximum girder lengths for a pretensioned W83MG and W95MG girder section are 54.6 and 51.0 m (179 and 167 ft), respectively. Greater spans are possible by using the post-tensioned versions W83PTMG and W95PTMG (see Fig. 6).

These girders have 200 mm (7.87

* In Reference 1, Sections W83MG and W95MG are designated as W21MG and W24MG, respectively, for their depth in hundreds of millimeters. WSDOT decided to revise these designations to reflect the approximate depths in inches. This style is consistent with the current standard girder designations.

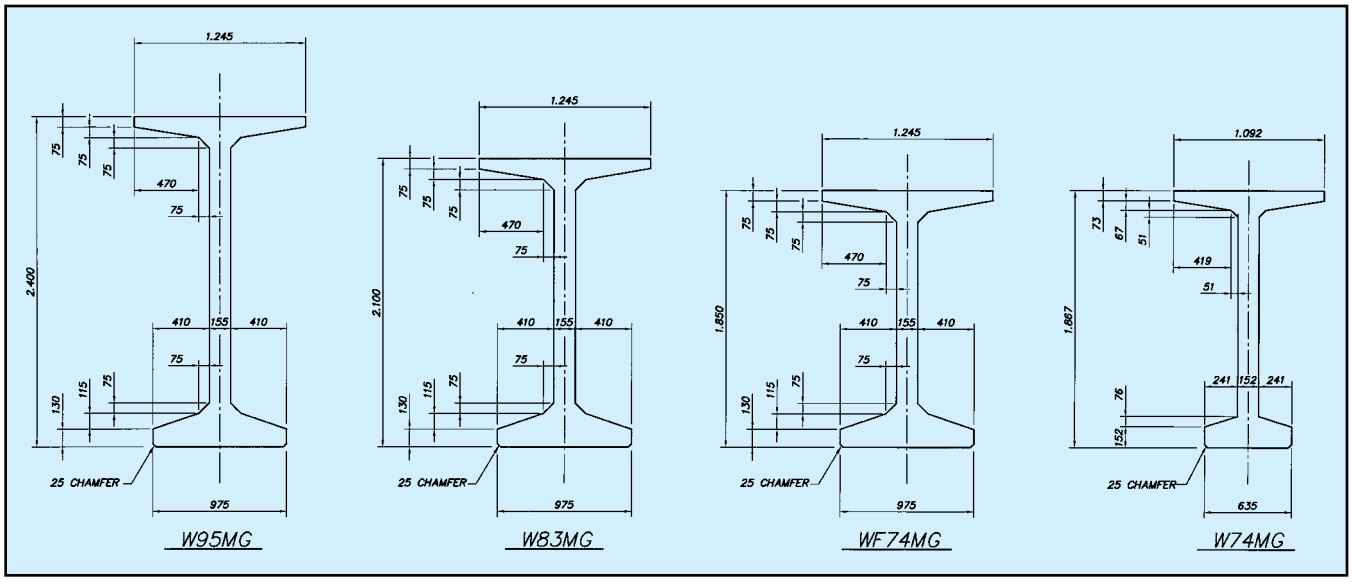

Fig. 5. Cross-sectional dimensions of W95MG, W83MG, WF74MG, and W74MG girder sections.

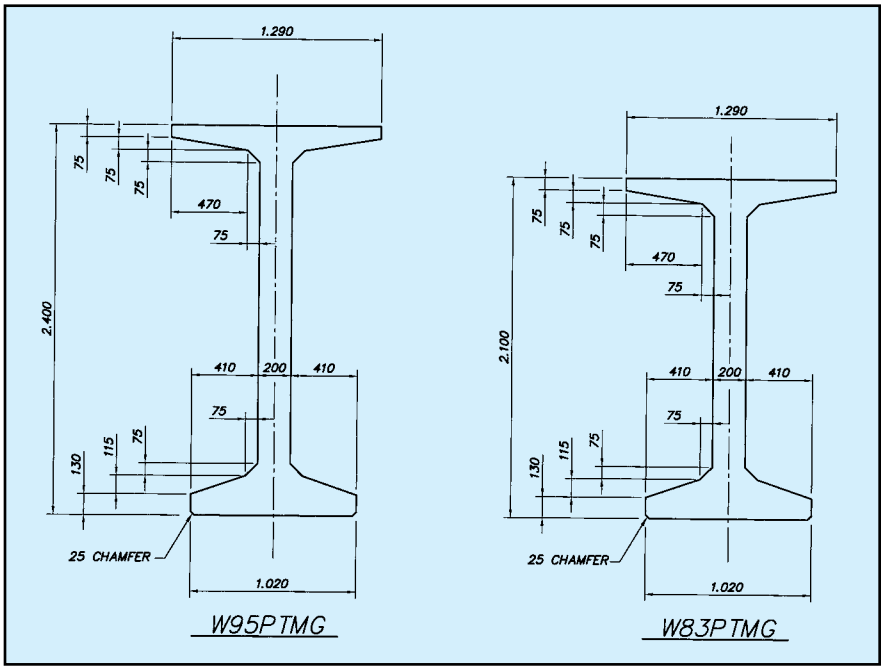

Fig. 6. Cross-sectional dimensions of W95PTMG and W83PTMG girder sections.

in.) wide webs as compared to the 155 mm (6.10 in.) wide webs of the pretensioned girders. Simple spans of up to 61.0 m (200 ft) for W83PTMG and 70.0 m (230 ft) for the W95PTMG girder are possible by shipping these girders in segments and post-tensioning them together at the project site.

PRECONSTRUCTION MEETINGS

Prior to the start of the design, WSDOT bridge designers met with all the stakeholders to discuss the concept of using the new W95PTMG girder section for this project. Several meetings were held with precast fabricators, contractor members of AGC, trucking industry representatives, WSDOT construction engineers, and state motor carrier officials to share their concerns and to identify any major flaws in the design and construction planning. This collective involvement ensured that everyone's expertise was included and that all parties would contribute to the success of the project.

One issue on the table was whether to post-tension the three girder segments prior to placing the roadway deck or to post-tension them after the deck was placed. If the girder segments were to be post-tensioned off the site and then erected, the girder segments would require a 28-day concrete strength of 69 MPa (10.0 ksi), and the closure pours at the splice joints would require a strength of 52 MPa (7.5 ksi). In addition, four 110 mm (4.33 in.) diameter ducts with nineteen 15.24 mm (0.6 in.) diameter low relaxation strands (AASHTO M203, Grade 270) would be needed.

By post-tensioning the segments after the deck was placed, one less tendon was required, and the required concrete strengths for the girders and closure pours were reduced to 55 and 35 MPa (8.0 and 5.0 ksi), respectively.

One of WSDOT's primary concerns was the quality of the cast-in-place splice joint. The solution with the lower strength demand was chosen, and the precast manufacturer bore the responsibility for the quality of the concrete in the joint.

Precast industry representatives offered three recommendations to improve girder fabrication and bridge construction efficiency. The first was to eliminate the thickened girder web end blocks so that the fabricators would not have to build special forms for the end region and could instead cast a girder with a constant web width. The second, which was necessitated by the first, was to place the post-tensioning anchorages in the cast-in-place transverse end diaphragms. The third suggestion was to use only straight pretensioning strands in the three precast segments.

The trucking industry representatives raised issues ranging from hauling permit fees to the additional costs of the Washington State Patrol escort-

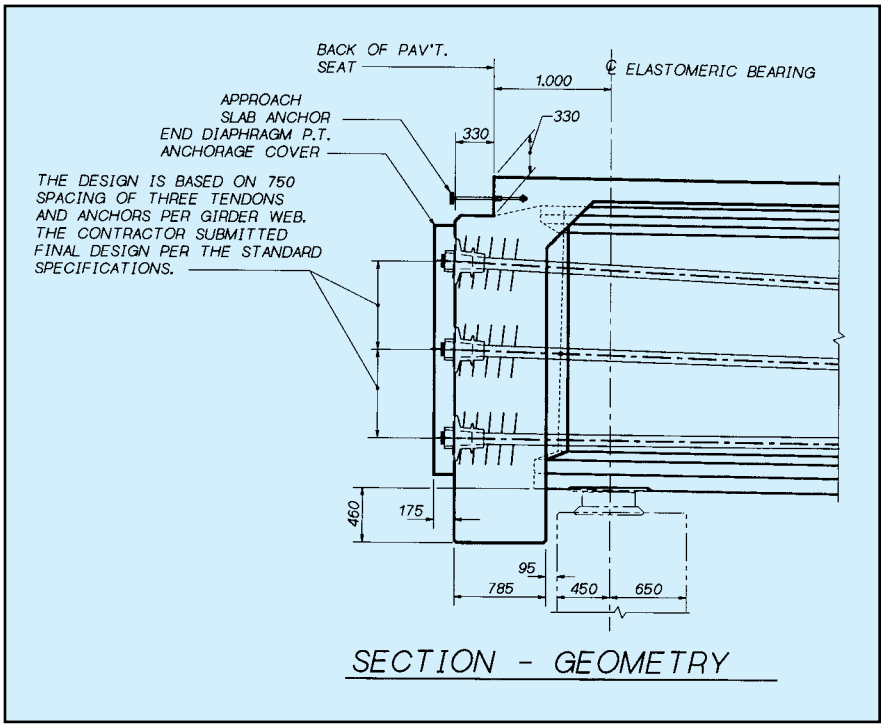

Fig. 7. Section through end diaphragm at post-tensioning anchorages.

Fig. 8. Reinforcement and post-tensioning anchorages at end diaphragm.

Fig. 9. Setup of middle girder segment.

ing loads to the bridge site. WSDOT agreed to do a route reconnaissance to ensure that, during transit, the girders would have adequate vertical clearance under existing bridges. Where required, WSDOT deep girder contracts include a special provision describing unacceptable routes and trucking permit requirements.

DESIGN AND ANALYSIS

The design concepts and details of the analysis for the girders, end diaphragms, and girder bearings are discussed in this section.

Girder Design

The superstructure design represented WSDOT's first post-tensioned LRFD effort. The concrete compressive stress was limited to $0.45f'_c$, with tension limited to zero for all load cases. Prestress losses were calculated using the refined method. Mathcad and BDS were the only software programs used. Girder closures were limited to regions outside of the maximum moment areas. High performance concrete was used since the required concrete strength exceeded 52 MPa (7.5 ksi).

Spliced girder design procedures are yet to be well defined in the AASHTO Specifications, with the bridge type falling between the AASHTO LRFD and the AASHTO Segmental Guide Specifications. Designers relied on WSDOT's considerable experience with spliced U-shaped girder design. AASHTO code issues like this one are currently being addressed in the ongoing NCHRP 12-57 research project "Extending the Span Ranges of Precast, Prestressed Concrete Girders."

The bridge has a calculated vertical frequency of 1.6 Hz and compares well with measured vertical frequencies obtained by Dusseau and Dubaisi[4] (on prestressed girder bridges along Interstate 5 in 1990). Fatigue is, therefore, not a problem with the unusually long simple-span prestressed girder.

End Diaphragm Design

The post-tensioning anchorages were located in cast-in-place end diaphragms that formed semi-integral abutments (see Fig. 7). Due to the depth of the girder, conventional jacking corbels would have been large and

Fig. 10. Forms were installed with a visqueen moisture barrier.

Fig. 11. Completed middle segment.

heavy. A smaller, compact end diaphragm was detailed with simple horizontal and vertical reinforcement mats (see Fig. 8).

Since the design required a discontinuity in the general zone at the girder-diaphragm interface, a strut-and-tie analysis was required by the LRFD Specification. The design was checked by the finite element method, which validated the stress trajectories assumed for the strut models and revealed a stress concentration at the girder-diaphragm interface. In response to this behavior, a small vertical fillet was detailed to smooth the flow of forces at the interface (see Fig. 7).

Bearing Design

In the mid-1960s, WSDOT introduced the semi-integral bridge as a replacement for integral concrete bridges. This concept was developed to eliminate expansion joints for short- to medium-span bridges. Semi-integral bridges accommodate displacements and rotations between the superstructure and end abutment with expansion bearings. This technique is used for concrete bridges with an overall length less than 107 m (350 ft).[2]

Fig. 7 shows the semi-integral abutment, end diaphragm, and bearings used for the Twisp River Bridge. In the past, all WSDOT-designed post-tensioned bridges were built with sliding bearings to accommodate superstructure movements due to temperature change, elastic shortening, creep, and shrinkage. Over the past decade, however, WSDOT has made a concerted effort to simplify bearing design wherever possible and has used high-load elastomeric bearings on several recent bridge projects.[3]

For the Twisp River project, it was felt that the movements could be accommodated by reinforced elastomeric bearings under each girder. Each girder is supported by 375 mm long by 750 mm wide by 170 mm high (14.76 x 29.53 x 6.70 in.) elastomeric bearings, which were designed by AASHTO Method B. The service dead load was 1270 kN (285.5 kips), the live load was 360 kN (80.9 kips), and the average compressive stress was 5.80 MPa (840 psi).

The contract special provisions required the contractor to perform long-duration compression load tests in accordance with the AASHTO Specifications. Further testing within a "lot" would be required if one of the test bearings failed to meet the test requirements. In addition, the contractor was required to raise the girders and replumb them if the upper and lower surfaces of the bearings were more than 25 mm (1 in.) out of plumb after post-tensioning.

Fig. 12. Girder end details showing two recessed abutment ends.

GIRDER FABRICATION

As previously discussed, the precast manufacturers requested that WSDOT consider placing the post-tensioning anchorages in the cast-in-place end diaphragms, thereby eliminating the need for special end blocks on the girders themselves. Harped pretensioned strands were also to be avoided because these would inevitably interfere with the post-tensioning ducts.

In complying with these requests, WSDOT produced a girder segment design that was so simple to construct that there were few problems with the fabrication of the girders.

Fig. 9 illustrates the simplicity of the design of one of the middle girder segments. As with all segmental, post-tensioned girders, the most critical issue was control of the location of the post-tensioning ducts, particularly where they exit the ends and must match up with ducts from the adjoining segments. Fig. 10 shows the formwork and visqueen moisture barrier in place. The forms for these girders are insulated and heated electrically, so the use of heavy curing tarps is not necessary. Fig. 11 shows a completed middle girder segment.

One unusual detail for the girders was the recess of the abutment ends to accommodate a cast-in-place fillet. Fig. 12 shows two recessed abutment ends sitting next to two ends that were spliced in the field. The finite element analysis of the post-tensioned general zone showed a stress concentration at the interface of the girder and end wall.

The solution was to provide a 200 x 200 mm (7.87 x 7.87 in.) fillet between the girder end and the end wall. Since the precast formwork could not accept the flare of the fillet, a recess in the end of the girder was provided by attaching wood blocking to the standard end plate. The fillet was then formed in the field along with the cast-in-place end wall.

Another issue that occurs infrequently on WSDOT projects is that of debonding straight strands at girder ends. Even though the practice of debonding strands is common in many parts of the country, it is not popular in the Pacific Northwest because of the inherent weakening of the girder end and other potential problems. Stresses at the girder ends are normally controlled by harped strands. In this case, however, the presence of the post-tensioning ducts precluded harped strands; therefore, some debonding was necessary.

Fig. 13 shows the strand locations where debonding was specified on the 30.9 m (101.4 ft) long middle girder segments. The length of

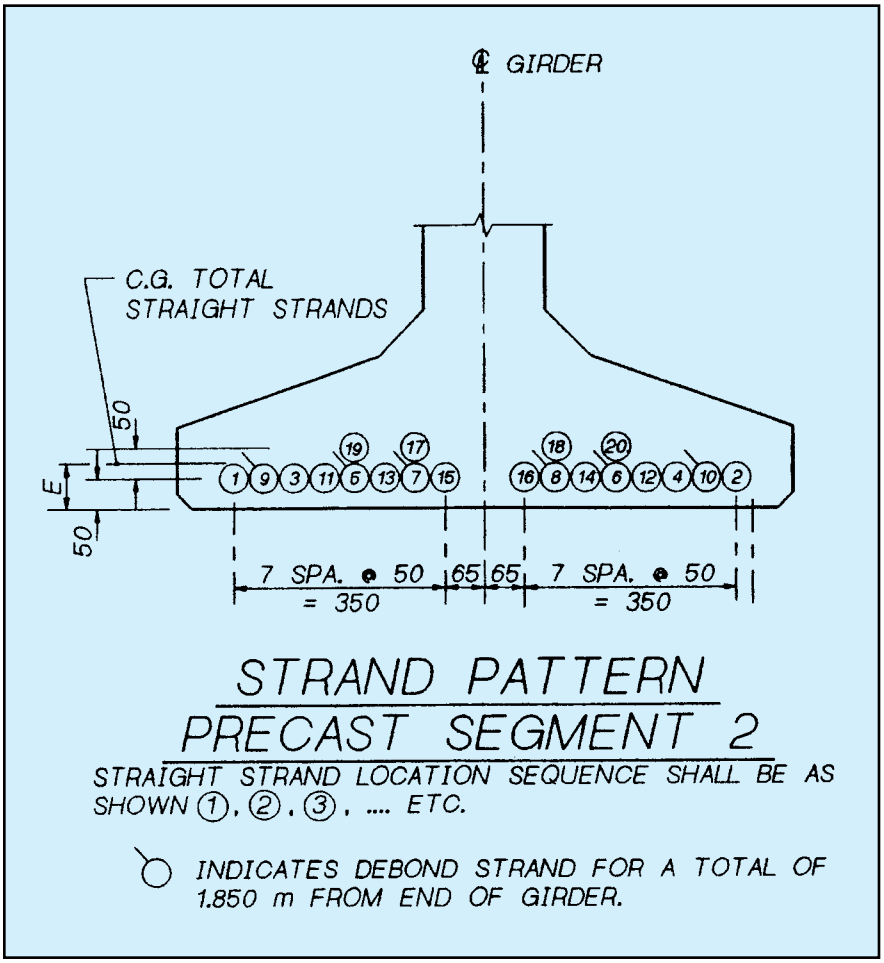

Fig. 13. Debonded strand locations in the middle girder segment.

Fig. 14. Girder segments sitting on temporary falsework bents.

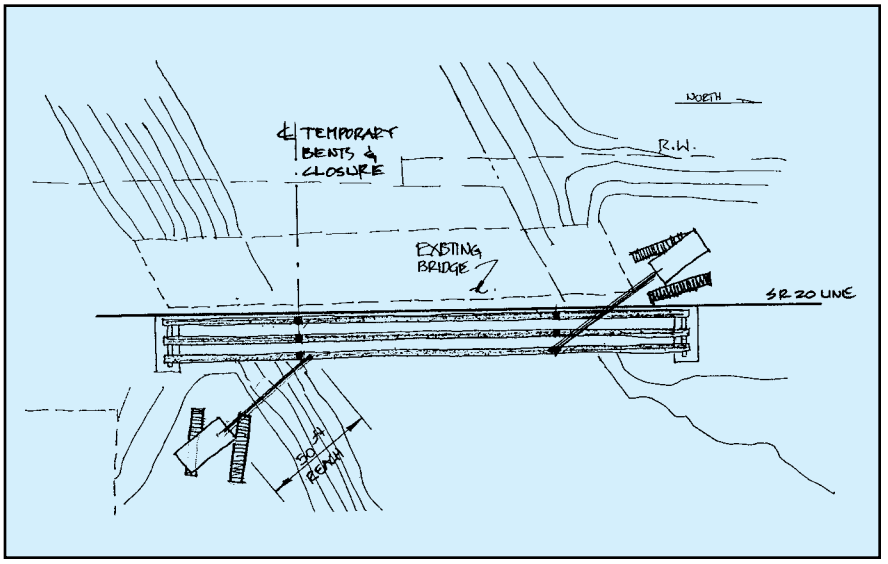

Fig. 15. Preliminary crane locations – Stage One.

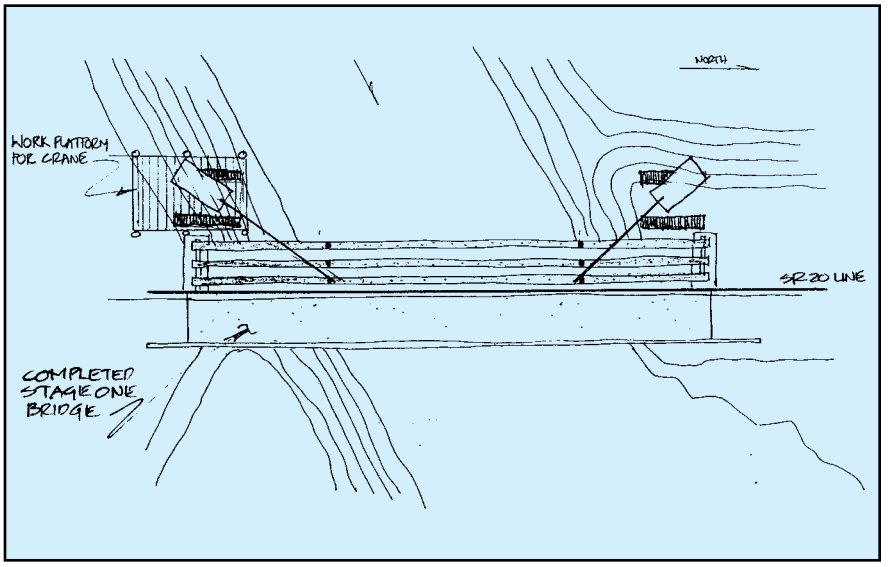

Fig. 16. Preliminary crane locations – Stage Two.

debonding was 1.85 m (6.07 ft), and split plastic sleeves were used over the strands. These sleeves fit fairly tight to the strand, and although the strands were released hydraulically, some splitting cracks were observed at the girder ends on the soffit of the bottom flange. These cracks coincided with the locations of Strands 9 and 10 shown in Fig. 13, and were approximately the same length as the debonding sleeves. This splitting occurred in the middle segments only, and the cracks were generally hairline in width.

Engineers speculated that the cause of the cracking was radial pressure induced by Poisson's ratio or "unraveling" of the strand after release. This appeared to be particularly true of strands in the thin portion of the bottom flange. Future plans to mitigate such cracking include locating debonded strands in thicker portions of the bottom flange, or using larger diameter rigid sleeves to prevent the development of radial forces.

HANDLING AND SHIPPING

No major problems arose with regard to handling and shipping the girder segments to the project site. Standard lift loops of strand embedded in the concrete were adequate for handling purposes, and lateral stability was not an issue. Shipping was also accomplished with standard hauling equipment, although a route reconnaissance was necessary to ensure adequate vertical clearances. The carrier for the project was V. Van Dyke, Inc., under contract with the general contractor, One Way Construction.

The carrier did request that the 30.9 m (101.4 ft) long middle girder segments be supported at 6 m (19.69 ft) from the ends during shipping. This shortened the turning radius of the vehicle and eliminated the need for a pilot car. Shipping with cantilevers this long required that two temporary top strands be added to keep the concrete stresses in the girder within allowable limits. To prevent spalling due to racking of the truck, additional transverse reinforcement was provided in the bottom flange at the locations of the truck supports.

Normally, girders are supported near their ends where this additional reinforcement is already present for control of splitting and bursting forces from the pretensioning. The width of the bearing surface between the girder soffit and truck support was limited to 635 mm (25.0 in.) to prevent point loads from developing at the tip of the bottom flange.

CONSTRUCTION SEQUENCE

The bridge was built in two stages over two construction seasons of 2000 and 2001. One City of Twisp requirement was that traffic flow had to be maintained at all times during the bridge replacement. During Stage One, one-half of the bridge was constructed, and the existing bridge was used as a detour. After the first stage was completed, traffic was rerouted to the new bridge so that the old bridge could be removed. The second or final stage of construction was completed during the following year, with one lane of light controlled traffic allowed on the Stage One structure.

The three girder sections were erected on temporary falsework bents by the general contractor (see Fig. 14). The middle girder sections weighed

607 kN (140.7 kips) and the end sections weighed 276 kN (62.0 kips). Next, the contractor placed the concrete for the girder closures, transverse intermediate diaphragms, and the bottom half of the end diaphragms. Finally, concrete for the roadway slab and top half of the end diaphragms was placed. Post-tensioning and grouting of the tendons took place after the girder closure pours achieved a concrete strength of 24 MPa (3.5 ksi).

Grouting of the post-tensioning ducts proceeded according to WSDOT Standard Specifications using Type I or II portland cement, water reducing admixtures, and optional fly ash. Water reducers were limited to AASHTO M 194 Type A or D and contained no chlorides, fluorides, sulfates, or nitrates. The contractor chose Sikament 300 water reducer and no fly ash. The duct joints were wrapped at the girder closure pours in accordance with the fabricator's details and inspected on site by the fabricator; these performed well during grouting.

Various construction schemes were investigated during design. Post-tensioning without the deck in place may have afforded the contractor more options. The girder sections may have been spliced adjacent to the site and placed using a launching truss.[5] However, the required closure pour strengths would have been 52 MPa (7.5 ksi) and difficult to place in a remote location.

Possible crane locations, reaches, and sizes were investigated during design to verify the feasibility of erection. The construction scheme that was shown on the contract drawings assumed the contractor would pick up the new girders from the existing bridge during Stage One construction. Figs. 15 and 16 are sketches used during the "roundtable meetings" showing preliminary crane locations for the two construction stages. Figs. 17 and 18 show the completed bridge in the fall of 2001.

Fig. 17. Completed bridge in the fall of 2001.

LESSONS LEARNED

The following paragraphs detail the lessons learned from the Twisp River Bridge project in terms of the use of the new W95PTMG girders to construct this single-span, semi-integral bridge structure.

Girder Soffit Cracking

Longitudinal splitting cracks were observed at the girder ends along the soffit of the bottom flange (at Strands 9 and 10 in Fig. 13). In the future, debonded strands should be located in the thickened portion of the bottom flange, where radial stress can be better resisted by concrete confinement.

End Diaphragm Distress

Small continuous horizontal cracks developed at midheight on the inside face of the end diaphragms during post-tensioning. Speculation for the

Fig. 18. Closeup of completed bridge.

Fig. 19. Side view of Twisp River Bridge (picture taken in February 2002). Courtesy: Alex Young, WSDOT Bridge Architect.

causes of cracking revolved around restraint of the bottom of the diaphragm and the jacking sequence. The bottom of the diaphragm was placed against the soil, which may have restrained movement due to post-tensioning. Also, vertical deflection of the elastomeric bearings may have increased restraint against the soil, since the deck was placed after the diaphragm.

The differential movement between the top and bottom of the diaphragm may have caused what appear to be flexural cracks. Additionally, the tendons were stressed from top to bottom, compounding the effects of differential movement. Providing a clear gap at the bottom of the diaphragm and stressing from the center row of anchorages may have solved the problem.

Camber Predictions

Monitoring of deck elevations revealed a minor sag profile in several girders in the first few months of service. Even though typical precast, prestressed concrete girders tend to camber upward, negative camber has been found on some spliced girder projects. The effects of deck concrete shrinkage and temperature gradient can be significant in deep girder sections. This is another issue that is likely to be addressed in the ongoing NCHRP 12-57 research project "Extending the Span Ranges of Precast, Prestressed Concrete Girders."

Figs. 19 and 20 show finished views of the bridge taken this February 2002.

CONCLUDING REMARKS

This paper describes the innovative features used in designing and constructing the Twisp River Bridge.

The following conclusions summarize the principal concepts and highlights, and provide recommendations based on the lessons learned:

1. New 2.4 m (7.87 ft) deep WSDOT W95PTMG sections were used to construct a 60 m (197 ft) long single-span bridge. The bridge was constructed with minor environmental impact to the river. Construction was in two stages over a two-year period.

2. The girders were delivered to the site in three precast, pretensioned segments, erected on falsework bents, and post-tensioned after the roadway deck was placed. Post-tensioning after deck placement reduced the required girder closure concrete strength to 35 MPa (5.0 ksi), compared to 52 MPa (7.5 ksi) when post-tensioning the girders before deck placement.

3. High performance concrete with a strength of 55 MPa (8.0 ksi) was used for the girders. The girder closure pours had a strength of 35 MPa (5.0 ksi) at 28 days; all other cast-in-place concrete had a 28 MPa (4.0 ksi) strength at 28 days.

4. High-load reinforced elastomeric bearings were used at the ends of the girders. The bearings were designed by AASHTO Method B. This is the first use of elastomeric bearings by WSDOT in a post-tensioned bridge.

5. Post-tensioning anchorages were placed in cast-in-place concrete end diaphragms. Jacking corbels where omitted in lieu of simpler end diaphragm geometry, where steel reinforcing mats where designed by the strut-and-tie method and stresses were checked by the finite element method.

6. Consideration should be given to placing intermediate diaphragms at girder closures instead of continuing only the girder section through the closure.

7. Minor cracking occurred adjacent to the debonded strands in the thin portion of the girder bottom flange. Such cracking can be mitigated by relegating the debonded strand locations to the thicker, more confined portion

Fig. 20. Finished view of Twisp River Bridge (picture taken in February 2002). Courtesy: Alex Young, WSDOT Bridge Architect.

of the bottom flange.

8. Minor flexural cracking occurred in the end diaphragms after post-tensioning. Such cracking can be mitigated by providing a gap at the bottom of the diaphragm, and by sequencing the tensioning from the center anchorages outward.

9. The total cost of the bridge project was $1,021,170 or $1289/m^2 ($120 per sq ft). The cost of the precast girder portion of the project came to $175,000.

ACKNOWLEDGMENTS

This paper summarizes WSDOT's experience in constructing its first bridge using the new deep WSDOT W95PTMG girder sections. Many individuals and organizations contributed to this effort. The authors thank the Washington State Department of Transportation (WSDOT), Concrete Technology Corporation, and the FHWA for funding the project. The support of J. A. Weigel, WSDOT Bridge Engineer, M. Myint Lwin, former WSDOT Bridge Engineer, Charles C. Ruth, WSDOT State Bridge Design Engineer, and WSDOT bridge designers Nathan S. Brown and Geoffrey D. Swett are very much appreciated. The contributions of Terry L. Mattson, WSDOT Project Engineer, Mitch S. Reister, WSDOT Chief Construction Inspector, and Alex Young, WSDOT Bridge Architect, are also gratefully acknowledged.

The opinions and conclusions expressed in this paper are those of the authors. They are not necessarily those of FHWA, WSDOT, or Concrete Technology Corporation.

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Owner and Designer: Washington State Department of Transportation, Olympia, Washington

Precast Concrete Girder Manufacturer: Concrete Technology Corporation, Tacoma, Washington

General Contractor: One Way Construction, Inc., Sedro Woolley, Washington

Post-Tensioning Subcontractor: Avar Construction Systems, Inc., Campbell, California

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