Bridge Views

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Landmark in the Works: Novel Use of Post-Tensioning in a Highly Curved Bridge

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Fig. 1. An aerial photo of the Mountlake Triangle Pedestrian Bridge Project

The Mountlake Triangle Project (MTP) Bridge is a highly-curved 400-foot long cast-in-place (CIP) post-tensioned pedestrian bridge spanning over Mountlake Boulevard between the Seattle Sound Transit light rail station and the University of Washington (UW) campus in Seattle, Washington. Construction of the bridge is scheduled to finish in early 2014. It is anticipated that this daring bridge will become a landmark for both the UW and the City of Seattle.

Design Challenges

The horizontal geometry resembles a highly curved "X" in plan view (see Figure 1), including a forked superstructure at each end of the bridge. The vertical design constraints required the bridge to meet stormwater runoff requirements and match predetermined elevations for three sets of stairs, two elevators, temporary and permanent vertical clearances for traffic, and permanent clearances for pedestrians and bicycles. Steel is usually the option for highly curved bridges. However the client, Seattle Sound Transit, requested the use of concrete for its ease of maintenance. Typically post-tensioning (PT) is not preferred in highly curved bridges due to the difficulties associated with handling the large out-of-plane forces induced. However, for this bridge, post- tensioning was chosen because it allowed a shallow section that met the vertical clearance requirements and produced a high level architectural finish without concrete cracking associated with non-post-tensioned concrete bridges.

The 92-foot minimum horizontal radius of curvature required special PT analysis, detailing, and construction. Local out-of-plane forces caused by the PT can potentially result in the concrete in front of the PT ducts failing. The strut-and-tie method was used to analyze and design the local PT reinforcement. Post-tensioning on a structure with such a tight horizontal curve tends to produce variable PT stresses across the bridge section. These variable PT stresses across the bridge section and between the webs can cause transverse tension in the slabs and undesirable torsional effects if the PT jacking forces are not arranged properly. It was found that using PT forces at each web that were roughly proportional to the height of each web minimized the undesirable variable stress and torsional effect due to PT. Additionally, a staged construction analysis was used to determine the optimal jacking sequence to minimize the undesirable PT effects described, and this optional jacking sequence was used during construction.

With such an unusual geometry, simplified straight line models are incapable of capturing important aspects of the structural behavior of the bridge such as the effect of the post-tensioning on the global bridge response and the stress and load distribution across the bridge section. Complex 3D finite element structural modeling techniques were used to capture the static and dynamic behavior of the bridge.

As seen in Figure 2, in-span hinges were added to the bridge to divide the bridge into three more regular bridge segments, which improved the overall static and seismic bridge behavior and simplified the analysis and design. Uneven span arrangement (end span to mid span length ratio is 0.40) required the use of special design techniques to control live load uplift reactions at the pier and at in-span hinge bearings. Some of the special design techniques used to control uplifting included the use of vertical tie rods, mass concrete at selected box cells, and rigid radial diaphragms at the piers. Also, a special seismic design criterion was developed to satisfy bridge (displacement based) and building (force based) code philosophies for the spans supported by the UW light rail station.

Fig. 2. Identifying the in-span hinges.

Construction Challenges

A number of challenges to bridge construction were presented by the complex bridge geometry combined with PT and the constraints imposed by the structure location. The bridge is located adjacent to existing structures and utilities and is next to the Seattle Sound Transit light rail station, which is also under construction. Constructing drilled shafts next to an existing aging underground parking garage structure, a spread footing next to a 54" diameter water main, and several piers on top of the UW station required the use of special construction techniques and planning . For example:

- Soil was excavated to temporarily unload existing structures to allow them to resist construction equipment surcharge loads;
- Secant pile shoring walls and isolating flexible material layers were used to protect utilities; and
- Strategic phasing of the

construction of the connection of the bridge to the UW station was used to minimize schedule impacts on the two projects.

The bridge superstructure formwork consisted of a series of adjustable straight formwork modules with tapered shims and plywood panels forming the curve shape. The placement of the duct and web tie reinforcement was carefully done because this reinforcement was critical to resist the PT out-of-plane forces.

The design of the MTP Bridge has pushed the limits for the use of post-tensioning in highly curved bridges, demonstrating that with the proper analysis and detailing, durable and low maintenance post-tensioned concrete can be used for bridges that have traditionally been made of steel.

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Evaluation of Common Design Policies for Precast Prestressed I-Girder Bridges

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Introduction

The American Association of State Highway and Transportation Officials (AASHTO) Load Resistance Factor Design (LRFD) Bridge Design Specifications, hereinafter referred to as the AASHTO LRFD Specifications, prescribes the minimum design requirements for safe highway bridges. However, many bridge owners have adopted more stringent policies for the design of precast-prestressed girder bridges. These policies specify design requirements for section properties, allowable tensile stress, and continuity.

Fig. 1. Owner adopted conservative design policies were used to design 205 ft long precast-prestressed girders in Washington State

The most common design parameters influenced by more stringent requirements include span capability (maximum span length that can be achieved), girder spacing, and prestressing levels. The sensitivity of these design parameters to the owner-adopted design requirements were studied for precast-prestressed concrete bridge girders and this article describes the results of the study. Span capabilityand prestressing requirements were computed based on the minimum requirements set forth by the AASHTO LRFD Specifications for 6 ft and 12 ft girder spacing. Each of the more stringent policies was then evaluated individually to understand its effect on the design. The combined effect of all the design policies was also investigated.

Survey of Design Policies

A survey of state Departments of Transportation (DOTs) was conducted to gauge the extent to which bridge owners deviate from the requirements in the AASHTO LRFD Specifications. Bridge owners were asked their policies in three areas: section properties, allowable tension, and continuity. Of the 38 respondents, 76% of them design with gross section properties in lieu of transformed section properties. With respect to the question on allowable tension, 71% design to allowable tension permitted by the AASHTO LRFD Specifications. With respect to the question on continuity, 50% design precast-prestressed concrete girder bridges as simple spans for all loads.

Conservative design policy	Reduction in span capability (%)		Reduction in girder spacing from baseline girder spacing (%)		Increase in required prestress (%)	
	6 ft spacing	12 ft spacing	6 ft spacing	12 ft spacing	6 ft spacing	12 ft spacing
Gross section properties	1.8 - 3.0	1.9 - 3.0	9.7 - 12.6	6.5 - 8.7	5.7 - 8.0	5.7 - 8.0
Zero allowable tension	4.9 - 5.4	4.9 - 5.4	22.5 - 27.3	15.3 - 17.3	11.4 - 12.7	11.4 - 14.0
Simple span analysis	2.9 - 3.2	2.0 - 2.9	13.2 - 17.4	6.3 - 10.0	7.7 - 9.1	6.0 - 8.3

Table 1. Effects of more stringent design policies on span capability, girder spacing and required prestress

Sensitivity Study

Slab-on-girder systems composed of a cast-in-place concrete deck on Washington State DOT precast prestressed concrete wide flange (WSDOT WF) I-girders were studied. Girder depths range from 36 to 100 inches. Interior girders were analyzed for various bridge configurations consisting of 6 WSDOT WF girders. The bridge deck, with haunch build-up of 3 inches, was assumed to be 7.5 and 9.5 inches thick for girder spacing of 6 and 12 feet, respectively.

The maximum strength of girder concrete was assumed to be 7.0 ksi at release of prestress and 9.0 ksi in service. The strands are 0.6 in. diameter jacked to 75% of the 270 ksi tensile strength (Grade 270 strands).

The baseline designs use the most liberal provisions allowed by the AASHTO LRFD Specifications; transformed section properties, allowable tension in accordance with AASHTO LRFD Specifications' Table 5.9.4.2.2-1, and full continuity in accordance with AASHTO LRFD Specifications Section 5.14.1.4. Baseline designs were established by setting the girder spacing to 6 ft and 12 ft and then the span capability was computed for various levels of prestressing. Comparative designs were then carried out using each of the policies. Two design parameters were held constant and the value of the third that satisfies the LRFD design criteria was computed. For example, to determine the sensitivity of the section properties policy on girder spacing, the span length and prestressing where held constant while the girder spacing satisfying the design criteria was computed. These designs were compared to the baseline designs. The results are summarized in Table 1.

The results indicate that the design policy with the least impact on girder capability is the use of gross section properties. The allowable tension policy carries significantly more impact.

Combined Design Policies

Table 2 summarizes the results if all three conservative design policies are implemented relative to the baseline design. In the case of the baseline girder spacing of 6 ft spacing, the required girder spacing that results from the conservative design policies becomes narrower than the width of the girder top flange making the design unobtainable. In all cases, the increase in prestress requires concrete release strengths in excess of 7.0 ksi.

Benefits of Conservative Design Policies

The AASHTO LRFD Specifications recommends a minimum service life of 75 years. Conservative design policies leave a margin of safety for unforeseen demands over the life of the

Reduction in span capability (%)		Reduction in girder spacing from baseline girder spacing (%)		Increase in required prestress (%)	
6 ft spacing	12 ft spacing	6 ft spacing	12 ft spacing	6 ft spacing	12 ft spacing
10.2 - 11.1	10.0 - 10.6	46.2 - 52.5	29.6 - 33.6	28.6 - 34.0	28.6 - 30.9

Table 2. Effects of combined stringent design policies on span capability, girder spacing and required prestress

structure. Supporting reasons for the conservative design policies include:

1. Historical increase in live load: design live loads have increased over the past few decades from HS-15 in 1944 to HL-93 in 1994.
2. Increasing use of overload trucks: Many permitted overload vehicles cross precast-prestressed girder bridges. The reserve capacity due to conservative design practices allows prestressed girder bridges to withstand these vehicles. Commerce would be adversely affected if these overloads could not be safely and conveniently moved.
3. Increase in number of traveling lanes: Lane widths on some routes have been reduced from 12 feet to 10 feet to accommodate more traffic lanes. Reserve capacity allows these bridges to accommodate increased traffic demand without strengthening or other modifications.
4. Reserve capacity for girders damaged by over-height collisions: Over-height load collisions with prestressed girder bridges often result in broken strands. Prior to repairs, the reserve capacity of the undamaged girders helps to keep the bridge in service.
5. Uncracked concrete under service conditions: A zero tension policy ensures that prestressed girders remain uncracked for flexure under service load conditions, resulting in longer service life and reduced life cycle cost.

Conclusion

This study shows the sensitivity of span capability, girder spacing, and prestressing requirements to three common owner adopted design policies. These policies are more stringent than the minimum requirements set forth by the AASHTO LRFD Specifications. Span capability is the least sensitive and girder spacing is the most sensitive to the design policies. Designing based on gross section properties in lieu of transformed section properties has the least overall influence. Reducing the allowable tension stress has the greatest overall influence and has the greatest impact on girder spacing requirements.

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Welded Wire Reinforcement Utilized as Shear Reinforcement in Concrete Bridge Girders

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Use as Shear Reinforcement in Concrete Bridge Girders

Welded Wire Reinforcement (WWR) is seeing an increased use as shear reinforcement in bridge girders, such as AASHTO Type II, III, and IV-4, Bulb-Tee Girders and U-Girder Sections, due to its available increased strength and the reduction in time or labor cost to install the material in the precast beds. State departments of transportation (DOT) allow the use of welded wire shear and confinement reinforcement in girders and many have developed standards for its use.

Fig. 1. Veteran's Memorial Toll way - Girder placement

What is Welded Wire Reinforcement?

WWR uses a cold worked/drawing process to produce differing wire sizes while increasing the tension and yield strengths of the wire. Wires are then welded together by an electrical resistance welding process to form a grid pattern. Engineers can specify the desired spacing and wire sizes.

Wire sizes range from Smooth/Plain (W) wire; W0.5 to W45 and Deformed (D) wire; D1 to D45. The number represents the area of the wire divided by 100. For example, a D31 wire has a cross-sectional area of 31/100 or 0.31 square inches. Wire is typically drawn down to the third decimal place as well. For example, the actual area of a ½" bar or wire is 0.1963495 square inches[1]. The wire producers will draw the wire to 0.197 square inches to meet the actual area of a ½" bar/wire. Several state DOT Bridge Departments allow the use of 0.197 square inches of welded wire reinforcement as the actual area for a ½" bar/wire.

Fig. 2. Welded wire reinforcement immediately after exiting the welding process

Spacings of wires vary based on design requirements. For instance, at the end of the girder - in the confinement zone, some state DOT's require a larger amount of steel reinforcement closely spaced to satisfy the confinement requirements. Spacing of a D31 welded wires can be as small as 2 inches; this will be limited by manufacturer .

Wire Conversion Engineers convert steel reinforcing bars typically shown on state DOT standard drawings to WWR. Also, many states now have standard welded wire reinforcement drawings.

One method of conversion is to replace the shear steel reinforcement with WWR on an equal area basis to 60 ksi yield strength material that is the typical steel reinforcement yield strength used. This method does not take the higher allowable yield strength that welded wire reinforcement is produced to according to ASTM A1064. Wire size and spacing are according to the design drawings.

The other method is to determine the area of steel required by converting the 60 ksi yield strength to higher AASHTO and DOT allowed steel yield strengths of 75 ksi. Differing wire sizes and spacing are utilized to provide the required shear reinforcement.

Once converted, shop drawings such as the one shown in Figure 2 for Welded Wire Shear Reinforcement in bridge girders are prepared and once approved, the welded wire is then fabricated and provide to the precaster/prestresser in various lengths and wire sizes to satisfy design shear requirements. Also shown are holding wires to maintain the spacing and shape for placing in the precast beds. The length of the styles (sheets) is dependent upon the precast beds, the labor and equipment utilized in placing and moving the WWR styles in the plant.

The use of WWR allows for decreased production times in the precast plants. And reduced inspection times as well as reduced overhead costs.

Welded Wire Reinforcement (WWR) utilized as shear reinforcement is a viable solution to satisfying the design requirements of girder design as well as providing a potentially lower cost alternative to bridge construction.

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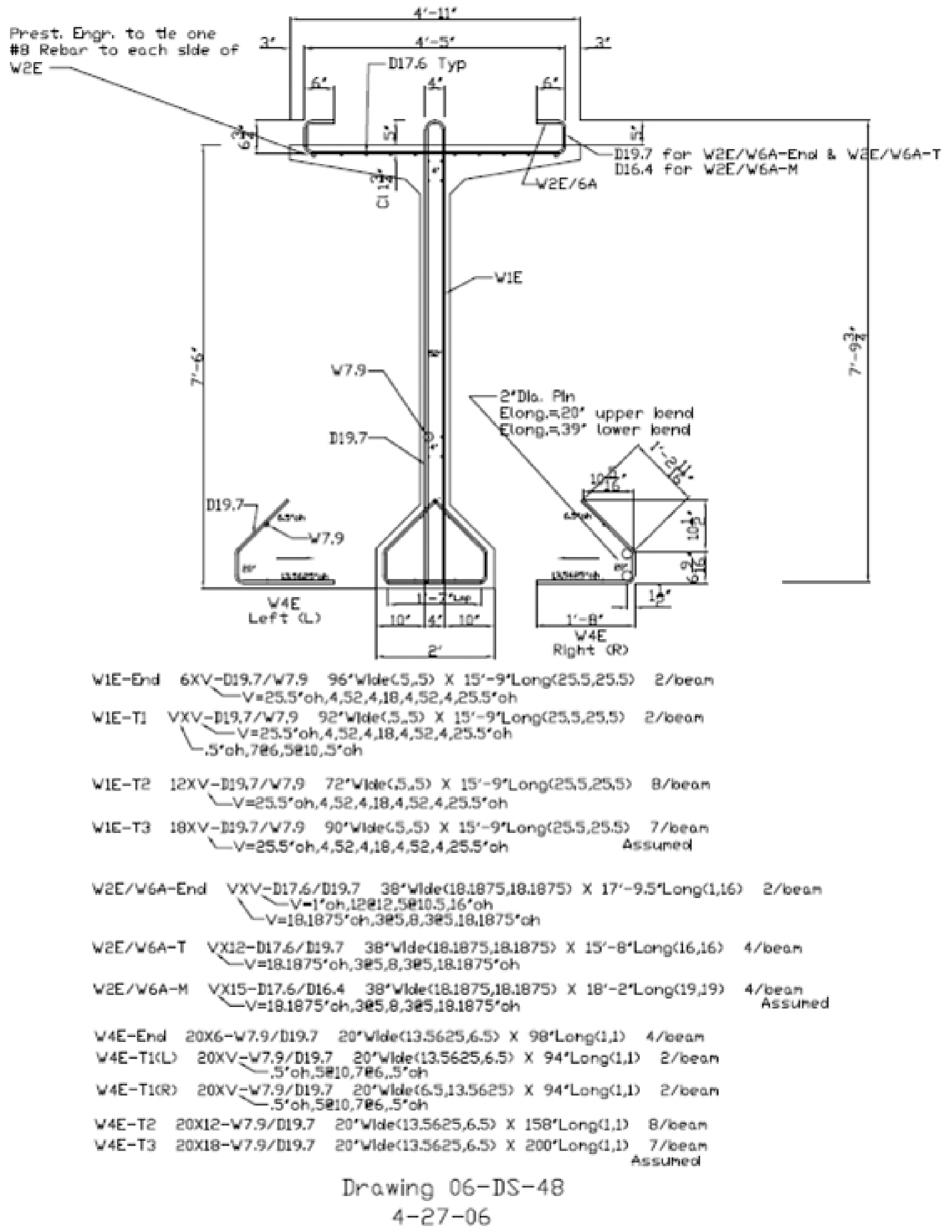

Fig. 3. Shop Drawing of Welded Wire Reinforcement (This drawing was prepared by Insteel Wire Products Company)

[1]Refer to ASTM A1064, "Standard Specification for Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete" for additional information and applicable requirements.