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Genesee Avenue Pedestrian Overcrossing, a Concrete Bridge with a High Degree of Curvature

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Fig. 1. Genesee Avenue Pedestrian Overcrossing (Rendering)

It is not every day that a bridge engineer gets the opportunity to design a semi-circular pedestrian bridge. Our opportunity presented itself as part of the 22-mile I-5 North Coast Corridor project located along the scenic I-5 corridor in coastal San Diego, California. The Genesee Avenue Pedestrian Overcrossing (POC) will provide a direct connection across Genesee Avenue for a new class 1 bicycle and pedestrian facility that will run parallel to I-5 on the west side. The two-span, 260-foot-long, 18-foot-wide, cast-in-place reinforced concrete box girder bridge was designed for an extreme horizontal curve (115-foot radius) as demanded by architectural and layout needs. The box girder type was selected for its high torsional rigidity. The bridge will be supported on a single-column bent and diaphragm-type abutments with deep foundations; this serves as a creative solution to handle the torsional demands imparted by the superstructure.

Aesthetics were a major consideration for this bridge being that this is the first significant project on the corridor and will be looked at to set the stage for all future segments. Close coordination with the City of San Diego, Caltrans and the San Diego Bicycle Coalition was required. The horizontal curve of the POC was ultimately chosen not only for the functional purpose of providing an uninterrupted pathway across Genesee Avenue for the benefit of the bicycle users, but also to provide a uniquely shaped signature structure for the coastal corridor. Integral colored concrete, a tapered octagonal column, pilasters behind each abutment, form liners on the approach retaining walls, and lighting integrated into the exterior concrete barrier face were all added to the final design of the project to complete the desired aesthetic theme. The metal railing is another unique architectural feature of the bridge consisting completely of weathering steel. It features a 5-inch-diameter steel tube bent to match the bridge radius, combined with perforated sheet metal components utilized to comply with strict American with Disabilities Act requirements related to pedestrian railings.

The unusual curvature of the POC required a high level of analysis for the superstructure that included development of a grillage model and a 3-D finite element model. The grillage model, used to design the bridge, was developed in accordance with the guidelines included in the NCHRP Report 620, "Development of Design Specifications and Commentary for Horizontally Curved Concrete Box-Girder Bridges"[1]. The grillage model consisted of longitudinal beams located along each girder line and transverse beams to model the bridge deck, soffit, and all diaphragms along the span. Since the maximum superstructure torque under dead and pedestrian loads could exceed the section's cracking torque, two boundary conditions were used to design for shear and torsion: a lower bound where the superstructure torsional stiffness is equal to 5 percent of the gross torsional stiffness, and an upper bound where the superstructure torsional stiffness is equal to 20 percent of the gross torsional stiffness. The shear and torsional reinforcement were designed for the envelope of these two conditions.

The 3-D finite element model was initially developed to confirm appropriateness of the grillage methodology, based on comparison of displacements between methods. In part, this confirmation method was suggested out of a peer review process that was elected by the designers to assist in evaluating the behavior of such an unusual structure. Thin shell elements were used to model the deck, girders, and soffit. The shell finite element method implicitly captures issues related to high degree of curvature including shear-flexure-torsion interaction, shear lag, and twist deformations due to distortion of the cells. The confirmation process not only provided support of the grillage method but allowed the engineers to debug and check parameters of the grillage design model. In addition, the complete independent check for superstructure design was then performed using the shell model.

The project was greatly accelerated in order to take advantage of additional state funding sources that became available as the planning phases of the project progressed. Due to the extreme horizontal curvature, this structure deviated significantly from what standard design methods can handle. A three-pronged approach that included an elective peer review process, varied analytical methods, and a full independent check enabled the design team to be successful in meeting this challenging deadline, while gaining confidence that adequate engineering was applied to the unique situation. The design of the Genesee POC was completed in July 2012. Construction is anticipated to start in 2015 with an expected completion early 2016.

Further Information

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References

[1]Nutt, Redfield and Valentine in Association with David Evans and Associates, and Zocon Consulting Engineers, 2008. Development of Design Specifications and Commentary for Horizontally Curved Concrete Box-Girder Bridges, NCHRP Report 620. Transportation Research Board, National Research Council, Washington, D.C.

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Controlling Prestress Release Cracks in High Performance Concrete Bridge Girders

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Fig. 1. An example of girder cracks

Very efficient new pretensioned concrete highway bridge girders have taken advantage of high strength concrete and unique shapes to allow the application of much higher levels of prestress. These newer girders often exhibit unexpected end cracking upon prestress release, a concern for designers and manufacturers. Cracks that are near prestressing strands may be particularly harmful if deck joints above the girder ends leak and allow chloride solutions to run along the girder ends. This could create a situation where chloride solutions penetrate the concrete surrounding the strands and cause accelerated corrosion.

Bulb-Tees are a prime example, often with deep sections, slender webs, and large amounts of prestress. The 54-inch deep girder shown in Figure 1 exhibits three types of typical end cracks: inclined cracks that run parallel to draped strands near the top of the web, horizontal cracks over the web depth, and bottom-flange Y cracks. The inclined cracks and horizontal web cracks may close as loads are applied to the girder and compressive end reactions develop. The Y or T shaped cracks, however, propagate into the bottom flange where strands are located and can increase in width to 0.05 inches as loads are applied.

End cracking is an endemic problem in prestressed concrete due to the highly localized force applied to the concrete at the release of pretensioned forces. Two mechanisms are involved: shear lag and localized bending. Both of these mechanisms can cause very high principal tension stresses in the concrete in the development length region at the girder ends, particularly with the slender cross sections of newer type girders.

Recent research conducted by the authors for the Wisconsin Department of Transportation (WisDOT) was aimed at controlling the tension stresses and strains developed in the concrete surrounding the strands to values below the tensile cracking strain. Both field testing and non-linear finite element analysis that simulated cracking were used to quantitatively examine various means of end crack control.

Crack control methods include: 1) providing vertical steel in the webs or spiral confinement reinforcing in the girder flange over the end transfer length region, 2) changing the drape of strands, 3) controlling the pretensioned strand cutting sequence during de-tensioning, and 4) debonding strands near the girder end.

Two types of steel reinforcing were examined for their effect in reducing concrete tension strains. Vertical steel bars, in addition to normal stirrup reinforcing, placed in the web of the beams are typically used to control the inclined cracks and horizontal web cracks. Results proved that only vertical bars very near the girder end (first two bars) are effective in controlling horizontal cracks. Using a bar size that would be difficult to fit in a web, 2 - #10 bars at 3 inches, reduced concrete tension strains in the horizontal crack region by 50% compared to typical girder reinforcing, but still allowed strains of more than double the concrete cracking strain to develop. Vertical reinforcing is effective in reducing crack widths for horizontal cracks, but not in eliminating them. Spiral confinement reinforcing in the bottom flange, likewise, is ineffective at controlling the Y cracks when placed around the bottom flange strands.

Reducing the strand drape or fanning the draped strands apart at the end of the girder can eliminate the inclined cracks, but does not eliminate the horizontal cracks. Changing the strand drape makes girders less efficient since it also requires a reduction in bottom flange strands to control initial end stresses.

Changing the sequence of stand cutting reduces concrete tension strains, but not enough to prevent Y cracks from developing.

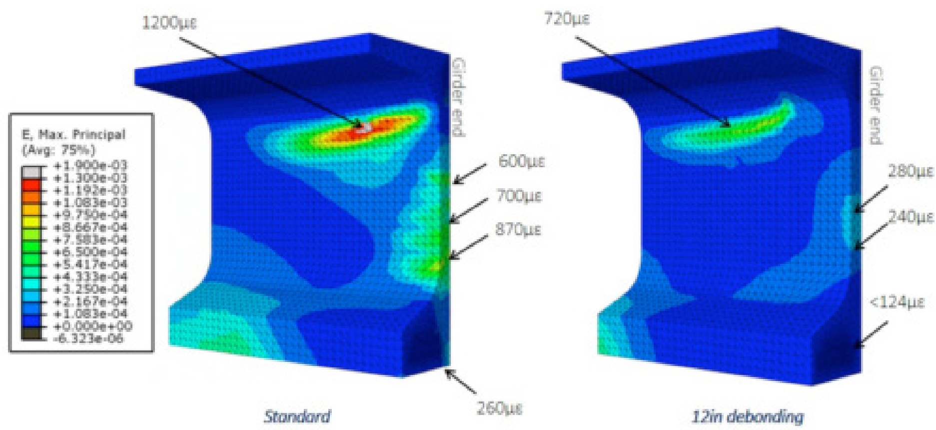

Fig. 2. Tensile strains for a 54-in. deep girder with standard strand configuration are compared to a girder where all strands are debonded for 12 inches

Debonding selected strands at the girder ends is effective in controlling or even totally eliminating girder end cracking. In examining WisDOT 54 inch, 72 inch and 82 inch deep girders with the full possible complement of strands, even debonding only 25% of the strands removed nearly all the concrete end cracks. Debonding 50% of the strand at the girder end completely eliminated cracking, but care must be taken to insure sufficient shear strength remains. AASHTO LRFD Specifications currently suggest that the total amount of debonding should be limited to 25% and that no more than 40% of the strands shall be debonded. Yet the AASHTO commentary acknowledges that some states have had success with greater debonding. An alternate solution: debonding all of the bottom straight strands for only 12 inches at the girder end also proved to be very effective in eliminating cracking and could be considered to satisfy AASHTO criteria if it occurred outside of the end bearing region. A simple solution to eliminate end cracking might be to move the girder bearing pads in 12 inches at the ends (using slightly longer girders) and debonding all the strand for 12 inches.

The results summarized below are aimed to help bridge engineers and manufacturers pick the best practices to control end cracking to extend the life of high performance prestressed bridge girders. In all of the scenarios below, except with debonded strands where draping is not needed, draped strands are assumed to be present.

Further Information

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References

Okumus, P., Oliva, M. G, "Evaluation of Crack Control Methods for End Zone Cracking in Prestressed Concrete Bridge Girders", PCI Journal, volume 58, Issue 2, pp 91-105. Okumus, P., Oliva, M.G., "Finite Element Analysis of Deep Wide-Flanged Pre-stressed Girders to Understand and Control End Cracking", Wisconsin Highway Research Program, Final Report, Report No. WHRP 11-06, June 2011.

Control method		Inclined Cracks	Web Cracks	Y Cracks
Increase in reinforcement area of	Two bars closest to girder end	Mild	Moderate	None
	Bars farther away from girder end	None	None	None
	Bottom flange stirrups	None	None	None
Debonding some strands at girder end		High	Moderate	High
Debonding all strands up to 12 in. from the end		Mild	High	High
Change in strand cutting sequence		None	None	Moderate
Draped strands	Removed	High	None	None
	Lowered	None	Moderate	None
	Lowered and spread	High	Moderate	None

Table 1. Practices to control end cracking

How to Specify and Construct Durable Crack Free Bridge Decks: Washington State Experience

Mark Gaines, PE, Washington State Department of Transportation, Mohammad Sheikhizadeh, PE, David Evans and Associates

The aging and deficient bridge infrastructure of the country is facing a replacement crisis. As funding for replacement of deteriorated bridges is becoming scarcer, there is a renewed interest by many states in constructing bridges that can provide long service life with minimal maintenance costs. Early age concrete bridge deck cracking is one of the major contributory factors affecting durability and loss of service life in bridges. Cracks in bridge decks can provide a direct pathway for water and salts to penetrate deep into the slab, potentially initiating slab reinforcement corrosion, and a potential process of rapid deck degradation.

Fig. 1. Bridge deck placement and curing

Over the past 15 years, the Washington State Department of Transportation (WSDOT) has become increasingly concerned with bridge deck cracking. In 1996, WSDOT made significant changes to the requirements for bridge deck concrete mixtures. Some of the key characteristics of this revised mix include:

- 660 lb/cubic yard cement (minimum)
- 75 lb/cubic yard fly ash (minimum)
- ¾" nominal maximum aggregate size

With a high cement content, these modifications resulted in concrete with excellent compressive strength (typically 6500+ psi at 28-days) and very low permeability. Unfortunately, this mix is prone to early-age shrinkage cracking. On recent projects constructed since 2005, both the size and the frequency of the shrinkage cracks have increased. It's difficult to determine exactly why deck cracking has become worse in recent years, but possible causes include changes in the chemistry or fineness of cement, variation in fly ash properties, and modifications in the chemical formulation of admixtures.

WSDOT has led research in recent years to determine the root cause of this problem and has used the results of research at Washington State University in field experimentation to control bridge deck cracking. Based on these efforts, WSDOT developed a new performance-based specification that has been used on approximately ten projects since 2011. Instead of placing requirements on minimum cementitious content, the new mix uses performance requirements, including:

- 28-day compressive strength of 4000 psi (minimum)
- 28-day drying shrinkage of 320 microstrains (maximum) (AASHTO T160)
- 56-day rapid chloride permeability of 2000 coulombs (maximum) (AASHTO T277)
- Scaling resistance, visual rating less than or equal to 2 after 50 cycles (ASTM C672)

WSDOT also made changes to the concrete placement and curing requirements. New requirements eliminate curing compound, instead requiring fogging to prevent moisture loss from fresh concrete. Once the finishing of the concrete is complete, presoaked burlap or other finishing blankets are immediately laid out on the wet deck. Soaker hoses are placed and deck is covered with reflective sheeting as soon as the concrete achieves initial set. After the 14-day wet cure is completed, diamond grooving is used to provide longitudinal tining of the deck necessary for skid resistance. These changes to the concrete mix and finishing/ curing practices have significantly reduced deck cracking as illustrated in Figures 2 and 3.


Fig. 2. (left) – View of the underside of a bridge deck constructed in 2010 with conventional bridge deck concrete. Fig. 3. (right) – View of the underside of a bridge deck constructed in 2013 using WSDOT performance-based bridge deck specifications

Although implementation of the new specifications has reduced deck shrinkage cracking, minor cracking is still evident. One factor that can significantly contribute to early transverse deck cracking is the differential between deck peak hydration temperature and ambient temperature. The strain due to this temperature differential can be as much as 300 micro-strains (Figure 4).

Performance-based deck concrete mixes with differential temperatures limited to 24°F have displayed significant reduction in transverse cracking. However, the Washington Association of General Contractors has expressed concern about including this limit in construction contracts due to unpredictability and the contractual risk it would expose them to.

WSDOT has been monitoring the hydration temperature signatures of performance-based concrete mixes and has found a remarkably high coefficient of correlation of R2 = 0.98 among concrete mixes in terms of initial hydration temperatures at different ambient temperatures and varying amounts of cementitious content (Figures 5 & 6).

The regression analysis holds true for mixes with up to 20% substitution with Supplementary Cementitious Materials (SCM), primarily Class F fly ash and slag cement. With the aid of the above graphs simplifying predictability of concrete mix hydration temperatures, it may become more acceptable to the contracting community to include limiting differential temperatures to 24°F in the specifications.

In summary, allowing performance-based concrete mixes with specific outcome requirements, best practices with curing and texturing, and considering the curing environmental factors in construction specifications can result in crack-free bridge decks.

For more information, please

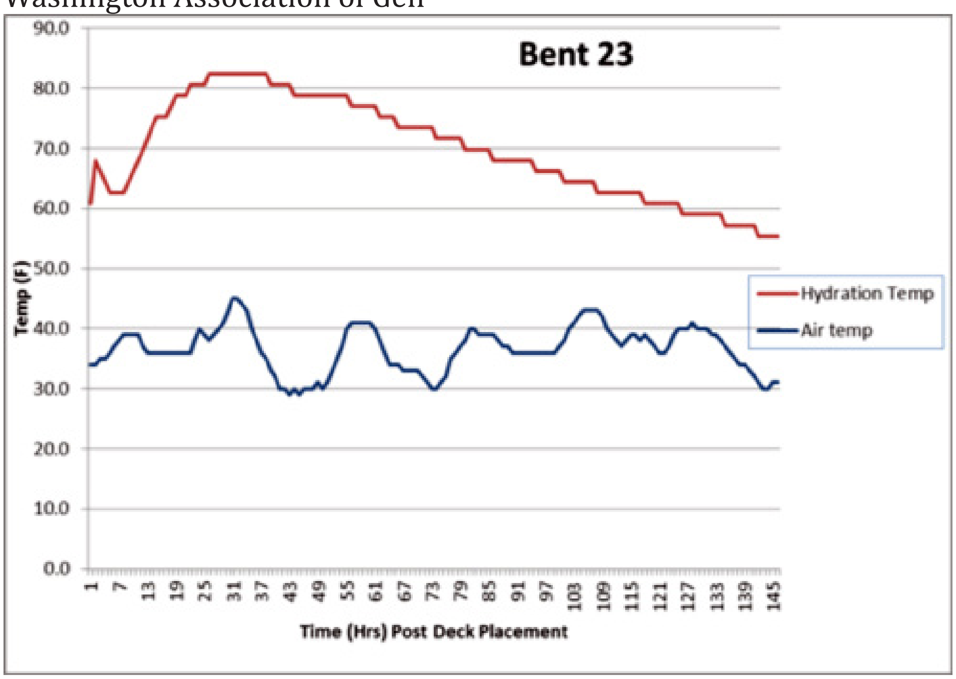

Fig. 4. Deck hydration temperature vs. ambient temperature

contact Mark Gaines of WSDOT at gainesm@wsdot.wa.gov, and Mohammad Sheikhizadeh of David Evans and Associates at mxsh@deainc.com

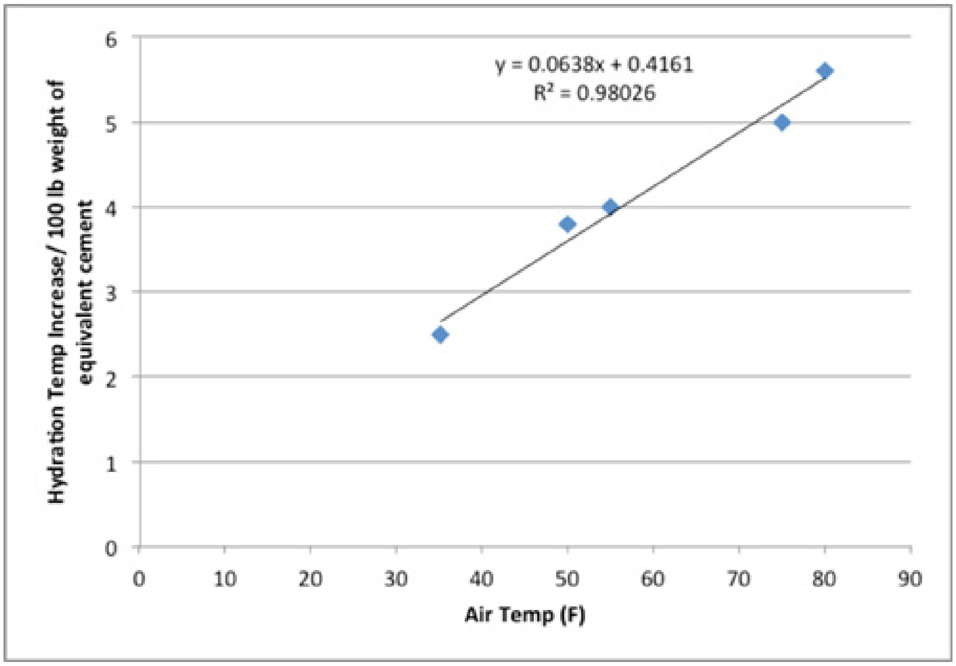

Fig. 5. Peak mix temperature rise prediction

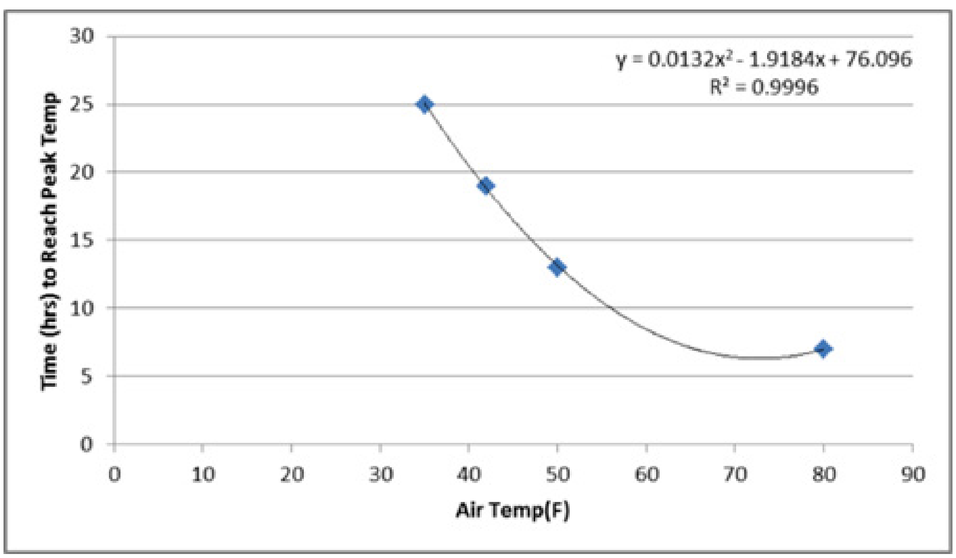

Fig. 6. Estimated time for deck mix to reach its peak temperature