*The Third International Bridge Seismic Workshop, 3rd IBSW*

*Seattle, Washington, USA - October 1st to 4th, 2019*

# LONGITUDINAL DECK JOINTS BETWEEN CONCRETE GIRDERS MADE USING UHPC

T. Peruchini[1], J. Stanton[2], P.M. Calvi[3]

[1] *Design Engineer I, Reid Middleton, Inc., Everett, WA 98204, tperuchini@reidmiddleton.com*
[2] *Professor, Department of Civil and Environmental Engineering, University of Washington, stanton@uw.edu*
[3] *Assistant Professor, Department of Civil and Environmental Engineering, University of Washington, pmc85@uw.edu*

## *Abstract*

The majority of short to medium span bridges built in the US today are constructed with precast, pre-tensioned girders and a cast-in-place deck. That system has proven economical and robust. In seismic regions, the cast-in-place deck serves to unify the superstructure into what is essentially a rigid body, which is then able to resist transverse seismic forces. Today, traffic congestion and access problems are causing DOTs to seek ever-faster ways of constructing bridges. One possible approach is to eliminate the deck casting operation, and instead to precast a segment of deck with the top flange of each girder. The resulting "deck bulb tees" must then be connected, flange to flange. The connection has, in the past, been made using welded steel inserts and/or grout keys, but these have not held up well under dynamic truck loading. An alternative is to add transverse bars that project from the precast top flanges, and to join them with a pour strip of Ultra-High-Performance Concrete (UHPC).

Much of the UHPC used today is proprietary, and is expensive. The study reported here describes the development of design rules based on a generic UHPC, which had been developed by colleagues at Washington State University. Tests were first conducted to establish the material properties of the UHPC. Then full-scale panel tests, which simulated a part of the joint in a real deck, were tested to confirm the splice requirements in the UHPC longitudinal joint. Analyses were also conducted to evaluate the force demands in the joint. It was found that a joint as narrow as 10" was sufficient to ensure fracture, rather than pullout, of the projecting bars, even though they were configured as non-contact splices. Such joints can therefore be used to ensure the integrity of the superstructure under seismic loading.

*Keywords: Ultra-high performance concrete; longitudinal joints; pre-tensioned girders; pre-cast bridge girders.*

## 1. Introduction

Prestressed concrete is the material of choice for bridge girders in many parts of the country. Such girders are economical, particularly from a life-cycle viewpoint, and they are also durable and need little maintenance. For many years, I-shaped girders with a cast-in-place deck have dominated the market on major highways, while totally precast girders such as deck bulb tees (e.g. [1]) have been used primarily on roads that carry low traffic, such as county roads, or where delivery of fresh concrete for the deck poses problems.
Efficient connections between adjacent deck bulb tees (DBTs) can be achieved through Ultra-High-Performance Concrete (UHPC) joints. An example of UHPC joint is illustrated in Figure 1.

Reinforcing bars extend laterally from the flanges, and are spliced in the longitudinal UHPC joint. Because of UHPC's very high bond strength, the splice length can be short and the joint can be narrow, even with epoxy-coated bars, thereby keeping the formwork simple and the costs down.

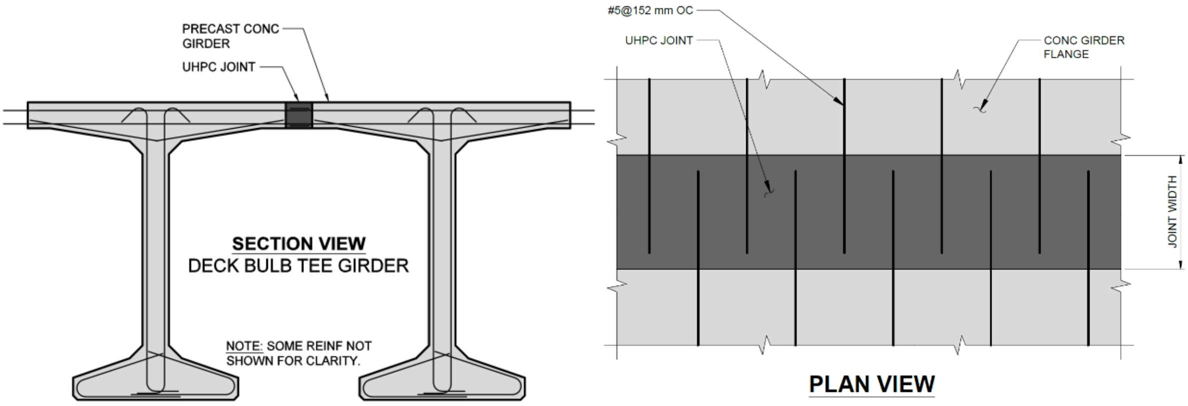



Fig. 1 – Deck bulb tee: conventional connection details

The work reported in this paper is intended to address those issues. The Project was sponsored by WSDOT and its primary goal was to use a locally-sourced UHPC and to develop corresponding connection details that would form a strong and durable connection between the flanges of the deck bulb tees. The mix design was developed by Washington State University [2], and the joint design and experimental verification were conducted by the University of Washington [3]. Thus, the design and performance of the joint are the main subject of this paper. Structural details not given here may be found in [3] and [4].

## 2. Numerical Evaluation of Demand on the Joint

The joint moments are caused primarily by wheel loading, and were investigated by conducting Finite Element Analyses. Those analyses were conducted using the program ABAQUS, and the deck was simulated using 8-node linear elastic shell elements. The purpose was to find the order of magnitude of the joint moments, rather than to conduct an exhaustive investigation of all possible geometries. The deck slab was modeled explicitly, including the taper near the girder webs, but the girders were represented by rigid supports to the deck. The loading was imposed by the standard AASHTO [5] wheel load corresponding to either a single axle truck, or to a tandem axle. The main approximations were:

- The girder webs were eliminated and were replaced by lines of rigid vertical support.
- In the longitudinal direction, the model was chosen to be as short as possible, to minimize the number of degrees of freedom, while still reflecting properly the important features of behavior. A length of three times the girder spacing achieved that goal.
- The model was run twice for each loading. In the "pinned" version, the slab was continuous over the supports that simulated the girders, but it was free to rotate there. In the "fixed" version, the slab

was also fully restrained against rotation at the girder lines. These two conditions represented respectively girder torsional stiffnesses of zero and infinity, thereby bracketing the true value.
- All analyses were linearly elastic.
- The gross concrete properties were used for all elements.

The analyses showed that:

- Of the AASHTO-specified loadings, the single-axle, rather than the tandem axle, gave the larger the response.
- The deck panels experience local membrane compression in the transverse direction, at the wheel location.
- The net transverse tension stress in the joint never exceeded 0.262 MPa (380 psi) when the service wheel loading was applied over the AASHTO-specified area of 250 x 500mm (10” x 20”) per double tire.

This tension stress is much too low to crack the UHPC itself, but it might initiate a crack at the interface between the UHPC and the adjacent deck panel. The possibility of interface cracking was evaluated experimentally in the test program.

## 3. Test Program and Experimental Setup

A series of Simulated Deck Panels were tested, using the rig shown in Fig. 2. Each consisted of two conventional concrete panels joined in the middle by a UHPC joint in which the bars projecting from the panels were spliced. One panel contained four bars while the adjacent panel contained three bars to form the splice. The specimens thus represented a 610mm (2-ft) wide transverse slice out of a deck panel. They were tested upside down with a 2.44m (8 ft) simple span. The central load was applied upwards, so that the tension face and crack patterns could be observed visually and mapped by the digital tracking system. A lateral restraint system was provided at the joint. The goal was to provide lateral confinement stresses in the joint that would lead to the same plane strain conditions as exist in the prototype. The device consisted of threaded rods, plates and a manually operated hydraulic jack. The lateral displacement was maintained as near as practically possible to zero by manually adjusting the jack load at each increment of the vertical load.

The specimens were statically determinate so that the moment at the joint could be established from the measured loads. This arrangement differs from the prototype, which is highly indeterminate and in which the transverse moments are further affected by the variations in stiffness caused by the taper in the panel thickness near the girder web. If the test specimens had included the indeterminacies of the prototype, small differences in the boundary conditions would have had significant effects on the joint moments, and the true joint moments could not have been found with certainty from the measured loads.

The test matrix for the panel tests is given in Table 1. In Table 1, joint width refers to the transverse width of the UHPC joint between the faces of the adjacent girder flanges and clear cover refers to that of the tension-loaded rebar when the panel is bent. The lateral offset of the opposing bars in the splice is considered zero if the bars protruding from one panel are exactly half way between the bars in the connecting panel.

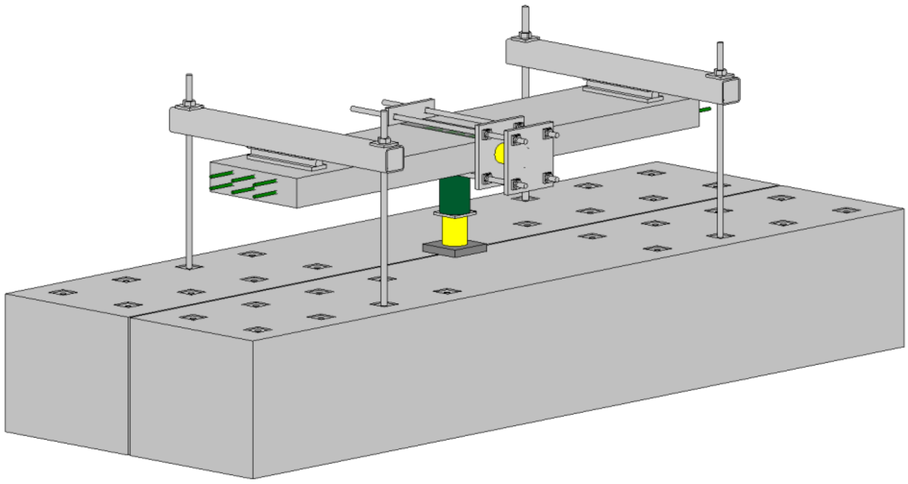


Fig. 2 – Panel test rig

Table 1 – Panel test matrix

| | Joint Width [mm] | Clear Cover [mm] | Rebar Lateral Offset [mm] | Deck Test Age [days] | Material Test Age [days] |
|---|---|---|---|---|---|
| Specimen 1 | 178 | 25 | 0 | 16 | 14 |
| Specimen 2 | 76 | 25 | 0 | 18 | 14 |
| Specimen 3 | 127 | 25 | 0 | 11 | 13 |
| Specimen 4 | 127 | 25 | 51 | 12 | 13 |
| Specimen 5 | 127 | 25 | 25 | 9 | 11 |
| Specimen 6 | 152 | 25 | 0 | 10 | 11 |
| Specimen 7 | 127 | 44 | 0 | 13 | 15 |
| Specimen 8 | 152 | 44 | 0 | 16 | 15 |

Supporting tests were also conducted. They included a series of material tests, to determine the properties of the UHPC itself, and a series of “curb” tests to investigate the properties of a tension splice that represented the tension side of the flexural splice used in the panel tests and the prototype. The material tests consisted of:

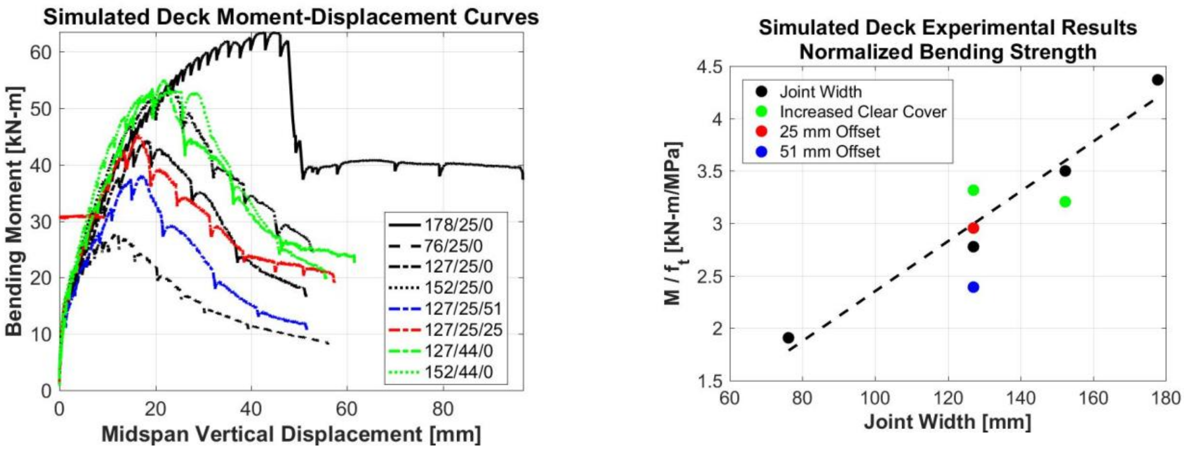
- Compression cylinders (100x200 mm, or 4”x8”, ASTM C39)
- Compression cubes (51x51x51 mm, or 2”x2”x2”, ASTM C109)
- Split cylinder tension tests (100x200 mm, or4”x8”, cylinders, ASTM C496)
- Flexural beam tests (75x100x150 mm, or 3”x4”x16” beams, ASTM C78)
- Direct tension tests (No ASTM)
- Bond tests (similar, but not identical, to ASTM D7913)

The results of the material tests are not discussed here, but can be found in [3].

## 4. Test Results and Analysis

The results of the Simulated Deck specimens are provided in Fig. 3 and indicate that the strength was greater with (i) a wider joint and (ii) a smaller bar offset. The wider joints also led to larger displacements at peak load, implying greater ductility. The greater strength from the wider joint is self-evident, but the fact that a non-contact splices is stronger than a contact splice of the same length is the opposite of what is expected in conventional concrete, and underscores the fact that UHPC splices behave differently. Only two offset tests were conducted and the maximum offset reduced strength by approximately 14%, but the trend in the two tests, shown in Fig. 3, was consistent. In the field, placement tolerances will affect the bar offset that can be achieved, which will, in turn, slightly affect the splice length needed. Increased clear cover had no discernable influence.

The typical failure mechanism displayed splitting of the UHPC similar to that that caused failure in the bond curbs. Vertical cracks initially formed through the clear cover on the top surface along the bars, but did not cause failure. The steel fibers in the UHPC were able to bridge these cracks and provide additional strength and ductility to the joint until a horizontal split formed through the plane of the extreme tension layer of bars.

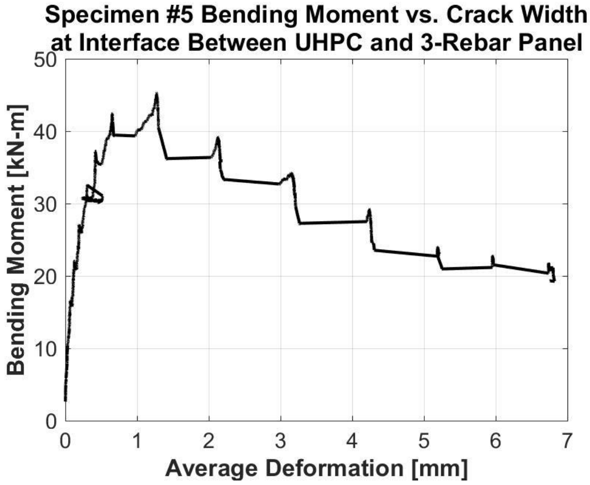



*(a) Moment vs. Displacement* *(b)Peak Moment vs Joint width*

Fig. 3 – Panel test results

During each experiment, no flexural cracks were seen in the body of the UHPC. All of the transverse cracking was concentrated at the interface between the UHPC and conventional concrete, as shown in Fig. 4. These cracks were first observed at a bending moment of approximately 11.3kN-m (100 kip-in.), corresponding to an elastic bending stress of 4.8MPa (690 psi) in the concrete, calculated using the gross cross-section properties. The cracks always occurred adjacent to the panel with three projecting bars, and remained quite narrow (< 0.5mm or 0.02”) until the splitting cracks formed in a horizontal plane in the UHPC. The measured cracking stress of 4.8MPa (690 psi) is almost twice the maximum service bending stress demand caused by the wheel loadings described earlier, thereby providing a safety factor of about 2.0 against cracking at the interface. At the panel’s ultimate strength, these interfacial cracks opened to approximately 1.25mm (0.05”). Fig. 4b shows a typical plot of bending moment versus interfacial crack width, averaged along the 600mm (2 ft) length of the crack.

*(a) Simulated deck interface cracks*




*(b) Moment vs. interface crack width*

Fig. 4 – Example of moment vs cracking response

## 5. Joint Design

The joint must be designed to provide satisfactory performance in all respects. The two primary criteria considered were the "Equal Strength Criterion", according to which the splice should have the same flexural strength as the deck panel, and a serviceability criterion. It was found that the Equal Strength Criterion was more critical than any plausible service criterion, so the latter is not discussed here.

For a given bar size and spacing in the girder flange (19mm at 150mm, or #5 at 6", here), the critical features of the joint to be selected are the width, end cover to the bars, concrete cover to the tension face, and bar offset.

The Simulated Panel tests investigated all of these features except the end cover, but none of the tests quite achieved bar fracture because the tested splice lengths were so short. They did show that the face cover had negligible influence on splice strength, and that bar offset had some. The major outstanding issue is thus the joint width, which in these tests was always 50mm (2") greater than the splice length, because the bar end cover was kept constant at 25mm (1") for all bars. To find the splice length and joint width needed to cause bar fracture, it was thus necessary to correlate the splice strengths from the Splice Curb tests and the Simulated Panel tests. The splice length required to fracture a bar was identified as 130 mm or 5.11''. The process employed to obtain this estimate is described in detail in [3].

The effects of fabrication and construction tolerances must also be accounted for. If, for example, two adjacent girders both have sweeps of 25mm or 1" [6] but in opposite directions, the splice could be 50mm (2") shorter than as designed. To allow for this eventuality, the minimum joint width was increased to 225mm (9"), implying a 175mm (7") splice length. For girders with no sweep, this represents a splice that is 1.4 times longer than the minimum necessary, which is enough to counteract the strength reduction caused by any reasonable longitudinal bar offsets. If both the sweep and the bar offset are expected to achieve their worst values simultaneously, a slightly wider joint would be needed to satisfy the Equal Strength Criterion. However, the joint would almost certainly still satisfy all service load and strength criteria based on applied loads. A probabilistic evaluation of fabrication and erection tolerances lay outside the scope of this study. However, these arguments show that greater refinement is not justified in the calculation of splice strength and ideal joint width, because the question of tolerances creates a larger uncertainty.

## 6. Conclusions

The following conclusions were drawn from the study:

1. The joint design is controlled by the criterion that the joint should have the same flexural strength as the deck panel. That criterion arises from the interpretation used here of the AASHTO LRFD prescriptive requirement that the deck be "sufficiently connected to act as a unit", rather than from a particular loading. It governs over almost any service load design criterion.
2. Using the non-proprietary UHPC developed here, a joint width of approximately 225mm (9") was found to be sufficient to satisfy the criterion in (1) above. This width allows for a 25mm (1") sweep in the girders.
3. Bond is the critical performance characteristic for the UHPC material in the joint for this particular application. UHPC compressive strength higher than that of the conventional concrete in the deck panel is unnecessary. The real need is for "Sufficiently High-Performance Concrete", (particularly with respect to bond), or SHPC, rather than for UHPC.
4. The mix developed by [2] and used in these tests gives bond and splice capacity nearly comparable to those of the proprietary UHPC mixes available in the market place.
5. In these tests, bond and splice failure in the panels was initiated by cracking and spalling of cover concrete, yet thicker cover provided negligible benefit. Others have found that thicker cover does provide higher strength, so the question should be investigated further.
6. Shear capacity of the joint was not investigated experimentally, but the demand is very low (less than 35kPa, or 5 psi), and is most unlikely to control the joint design.
7. The effectiveness of a formed keyway in the face of the joint was not investigated experimentally here, but despite the smooth joint face used in these tests, no shear slip was detected in any of the tests. This lack of slip in the tests, and the low shear stresses to be expected in the field, suggest that a key is unnecessary.

## 7. Acknowledgements

The Washington State Department of Transportation sponsored the project, and that support is gratefully acknowledged, as is the cooperation with the research team at Washington State University (Profs Qiao and Allena).

## 8. References

[1] WSDOT (2018): Bridge Design Manual (LRFD). Document M23-50.18, *Washington State Department of Transportation*, Olympia. June. 1294 p.

[2] Qiao P, Zhou Z, Allena S (2017): Developing Connections for Longitudinal Joints Between Deck Bulb Tees - Development of UHPC Mixes with Local Materials. Report no 869.1, *Washington State Dept. of Transportation*. June.

[3] Peruchini TJ, Stanton JF, Calvi PM (2017): Investigation of Ultra-High-Performance Concrete for Longitudinal Joints in Deck Bulb Tee Bridge Girders. Report no 869.2, *Washington State Dept. of Transportation*. June.

[4] Peruchini TJ, Stanton JF, Calvi PM (2020): Longitudinal Joints between Deck Bulb Tee Girders Made with Non-proprietary UHPC. *ASCE Bridge Journal*, under review.

[5] AASHTO (2012). AASHTO LRFD Bridge Design Specifications. *American Association of State Highway and Transportation Officials*.

[6] Russell BW (2019): Sweep in Precast, Prestressed Concrete Bridge Girders. *Aspire*, Precast/Prestressed Concrete Institute, Spring, 38-41.