Ultra-High Performance Concrete in Ohio

Student Study

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16. Abstract

This research reviewed the current and future uses of UHPC in the state of Ohio for bridge related structures. The first objective was to review the performance of UHPC in the Sollars Road adjacent prestressed concrete box beam bridge in Licking County constructed in 2014. UHPC was utilized in the shear keys between the box beams and included dowel bars. Data acquired during 2017 from truck loading and thermal changes was nearly identical to that obtained in 2014 shortly after bridge construction was completed. This implied the bridge's UHPC longitudinal joints are performing very well three years after construction.

The research also monitored and evaluated the 2017 UHPC closure pour for the LIC 310-0096 bridge since this was a new usage of UHPC. Data was collected during placement and during different seasonal periods to monitor daily and seasonal thermal effects. Strains from temperature changes showed strains in the range of 150 to -400 microstrain. Minor hairline cracking was observed in the UHPC and adjacent conventional concrete deck.

The final objective of the research was to provide review and advisement to ODOT related to the GAL 160-18.84 UHPC deck panel connections. The deck panels were to be full width except near the bridge ends due to the high 35º skew. However, the Department ultimately directed to construct a traditional cast-in-place deck for reasons unrelated to UHPC.

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Executive Summary

Ultra-high performance concrete (UHPC) is a relatively new concrete material being implemented in the United States in a variety of applications for bridges. Originally, UHPC was used in superstructure members such as a modified prestressed concrete girders and unique sections. More recently, UHPC is frequently being used for bridge connections in precast concrete deck panel connections, joints of decked bulb tee girders, and keyways of adjacent prestressed concrete beams. Other relatively new uses of UHPC have been waffle deck slabs, deck overlays, stay in place formwork, link slabs to create continuity in continuous bridges, and as a repair material for the aging infrastructure.

This research reviewed the current and future uses of UHPC in the state of Ohio for bridge related structures. The first objective was to review the current usage and performance of UHPC in the Sollars Road adjacent prestressed concrete box beam bridge in Licking County constructed in 2014. This was the first adjacent prestressed concrete box beam bridge in the U.S. to utilize UHPC in the shear keys between the box beams. Diaphragms only existed at the ends of the bridge. Though a longitudinal crack was observed in the asphalt, this cracking was determined to be due to the seam between paving separate lanes. The crack existed above the center girder of the seven girders utilized in the bridge and not above a longitudinal joint. Data acquired during 2017 from truck loading and thermal changes was nearly identical to that obtained in 2014 shortly after bridge construction was completed. This implies the bridge's UHPC longitudinal joints are performing very well three years after construction.

The research also monitored and evaluated the 2017 UHPC closure pour for the LIC 310-0096 bridge since this was a new usage of UHPC. LIC 310-0096 was a continuous four span bridge project over IR70. The project included multiple phases and widening of the existing bridge. Phase 3c of the project involved a 9¼" UHPC closure pour between a portion of the modification of the existing superstructure (Phase 3a) and an additional shared used pedestrian path (Phase 3b). The joint had a single #5 longitudinal rebar in the top and bottom. Two additional #4 longitudinal bars existed in the top of the joint over piers. Transverse #5 bars were spaced at 5¾" in the top and bottom and were continuous through the UHPC closure joint. Instruments were installed within the closure pour and adjacent conventional deck. Data was collected during placement and monitored for a period after placement. Data was also obtained during truck loading and during different seasonal periods to monitor daily and seasonal thermal effects. Truck loading

did not show significant strains due to the size of the applied loading relative to the bridge. Strains from daily temperature changes showed strains in the range of 150 to -400 microstrain. This level of strains should not have caused any cracking issues. However, minor hairline cracking was observed in the UHPC and adjacent conventional concrete deck during late April of 2018.

The final objective of the research was to provide review and advisement to ODOT related to the GAL 160-18.84 UHPC deck panel connections. The three span continuous steel beam bridge had spans of 58 ft., 72.5 ft., and 58 ft. The deck panels were to be full width except near the bridge ends due to the high 35^{o} skew. The design resulted in a unique connection of the end panels over the girders. However, the Department ultimately directed to construct a traditional cast-in-place deck for reasons unrelated to UHPC. Details of the panels and connections are still provided for possible future usage.

UHPC has many advantages over conventional concrete or grout materials. It has a very high compressive strength typically exceeding 22 ksi. The compressive strength also increases at a rapid rate with 14 ksi typical in 2 to 3 days under normal ambient conditions. This rapid strength gain can be an advantage in accelerated bridge construction (ABC) projects where time is critical. The high compressive strength is accompanied by a high tensile strength near 1 ksi. UHPC has a high bond strength which shortens development lengths of reinforcement and results in improved interface behavior between UHPC and substrate materials (conventional cast concrete or precast concrete) in connections or repairs. UHPC is also flowable which allows it to enter tighter spaces and around reinforcement.

Disadvantages of UHPC include the very high relative cost compared to conventional concrete and grouts. Part of this higher cost is due to limited suppliers, but this is rapidly changing. The high cost is also the result of the material being new and unfamiliar to designers and contractors which often build in conservatism in designs and bids. Currently design specifications from AASHTO do not include criteria related to UHPC. However, efforts are underway by numerous organizations and groups to improve on documentation to assist designers, owners, and contractors.

Regardless of the UPHC usage, the owner and contractor should work closely with the UHPC supplier early in the project process and throughout the project so all involved can be well informed to assure success.

Project Background

The state of Ohio has the second largest bridge population in the U.S. with a total of 27,345 as noted in the National Bridge Inventory. Of these bridges, 58% are classified in good condition, 36% in fair condition, and 6% in poor condition. In order for Ohio to maintain the infrastructure for which they are responsible while constructing new economic infrastructure, ODOT must remain involved in new methods and innovative materials.

Ultra-high performance concrete (UHPC) is a relatively new material being implemented in the United States in a variety of applications for bridges. UHPC is a portland cement-based composite material that can be altered for each application. The material usually consists of portland cement, ground quartz, fine sand, silica fume, high-range water-reducing admixture, steel fibers, and water (Russell and Graybeal, 2013). The Federal Highway Administration (FHWA) defines UHPC as a "cementitious composite material composed of an optimized gradation of granular constituents, a water-to-cementitious materials ratio less than 0.25, and a high percentage of discontinuous internal fiber reinforcement." The mechanical properties of UHPC include compressive strength greater than 21.7 ksi and sustained post-cracking tensile strength greater than 0.72 ksi. UHPC has a discontinuous pore structure that reduces liquid ingress, significantly enhancing durability compared to conventional concrete (Graybeal, 2014). An important component in the mix of the UHPC is the steel fibers. The fibers allow the UHPC to maintain tensile capacity even after cracking (Yuan and Graybeal, 2014, 2015). The steel fibers are also considered important in reducing the shrinkage in UHPC because they provide an internal restraint (De la Varga and Graybeal, 2014). The superior properties of UHPC have led to a reduction in the development length of embedded steel reinforcement, which has allowed the field-cast connections between bridge elements to be smaller. The superior mechanical properties and durability of UHPC have led to its use as a grout material in bridge connections. The bulk of connections used to date have been deck panel connections by the states of New York, Iowa, Oregon, and Montana (Graybeal, 2014). UHPC has also been used in connections of deck bulb tee girder bridges in the states of New York, New Jersey, Massachusetts, and Idaho. Many applications of UHPC in infrastructure projects exist. The Federal Highway Administration has an interactive map showing projects across North America utilizing UHPC. A screen shot of the map is shown in Figure 1 and details of each project can be viewed at the interactive map by selecting a project site. The map can be found at:

http://usdot.maps.arcgis.com/apps/webappviewer/index.html?id=41929767ce164eba934d70883d775582.

The usage of UHPC in Ohio has been limited to a single project in Fayette County (Steinberg, et al., 2015) with the additional projects that were part of this research project. In order for Ohio to fully understand and reap the benefits that UHPC has to offer, it is imperative that the material is better understood by monitoring and measuring its field performance. This will allow UHPC to be used properly in future designs and applications. Some of the applications may not have been applied yet to date or even realized until designers become more familiar with the material and its attributes.

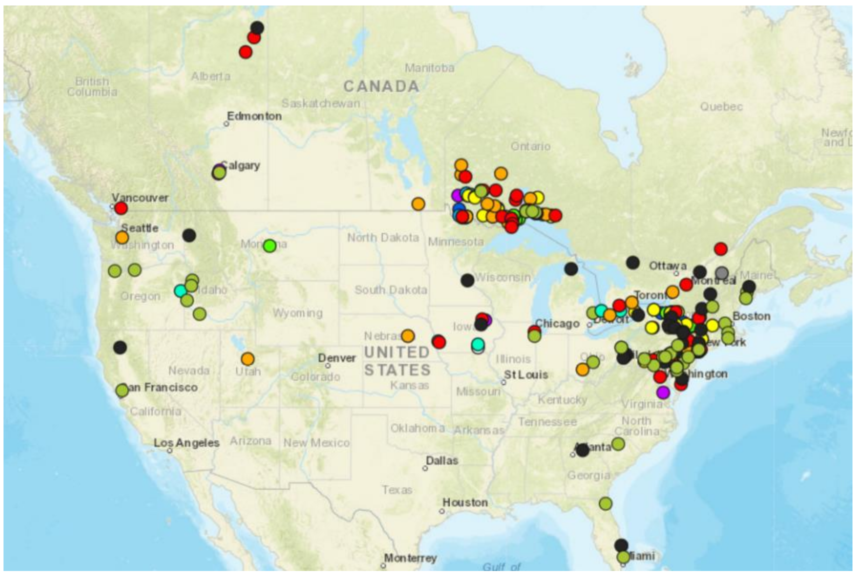


Figure 1: FHWA Interactive UHPC Project Map

Research Context

The primary goal of the project was to evaluate the usage of UHPC in Ohio. This was performed through three objectives.

1. Review the current usage and performance of UHPC in the Sollars Road adjacent box beam bridge in Fayette County constructed in 2014.
2. Monitoring and evaluating the UHPC closure pour for the LIC 310-0096 bridge since this was a new usage of UHPC for Ohio and the U.S.
3. Provide review and advisement to ODOT related to the GAL 160 UHPC deck panel connections.

A summary of literature related to each of these objectives is provided below. Additional information on UHPC can be found in Appendix A.

The Sollars Road Bridge in Fayette County, OH, was the first bridge in the United States to use a UHPC connection for adjacent box beams. Similar to the deck-level connections between precast concrete bridge deck panels, an improved structural connection that is capable of transferring shear, moment and axial tensile/compressive forces across the connection can be created with field-cast UHPC connections that bond to the precast interfaces and use a lap-splice reinforcement detail. In addition, UHPC connections for adjacent box beams do not require transverse post-tensioning or a structural concrete overlay. Since the completion of the Sollars Road bridge, Delaware, New York, Pennsylvania, and Michigan have completed adjacent box beam bridges using UHPC.

Published research on deck closure pours, such as being done on the LIC 310 bridge, does not appear to exist at this time. Therefore, this is a new usage of UHPC connection details. Part of the issue with closure pours is that there is often limited space for the connection which makes bond of reinforcement difficult. Some projects have used UHPC in similar types of connections such as expansion joint repair, live load continuity joints, link slabs and connections between approach slabs, but these cases are limited.

Adjacent precast deck panel connections as was proposed in GAL 160 include overlapping rebar from adjacent panels then filling the void between them with UHPC. The rebar is spaced typical to conventional deck design and transfers moment, shear, and tensile forces across the joint. The tension development length for rebar embedded in UHPC is significantly less than the length required in conventional concrete. Therefore, straight short lengths of rebar can be used which simplify the connection and consequently reduce reinforcement costs. Shorter lap lengths allow the connection width to be small, therefore minimizing the volume of UHPC and simplifying the formwork to create the connection. The details to connect prefabricated deck panels to girders require the use of shear studs or rebar extending from the girder into block-out pockets in the deck panel that can be filled with UHPC instead of grout to create a composite deck/beam structure. The thixotropic, self-consolidating properties allow UHPC to flow into the often confined and congested spaces associated with shear pocket details. The UHPC has good bonding ability to adjoining precast surfaces and the discontinuous pore structure nearly eliminates liquid ingress. This eliminates the need of an additional construction activity to place a deck overlay. UHPC also has the ability to improve internal stress distribution, thus enhancing the composite action between the prefabricated panel and supporting beam. Often the UHPC detail combines the panel-to-panel connection detail with the deck-to-girder connection detail into a single UHPC field-cast connection that runs along the girder line.

Currently, there are no design specifications in the U.S. for UHPC. To date, the most helpful document for UHPC connection design is covered in Technote FHWA-HRT-14-084 (Graybeal, 2014b). This document provides guidance and associated commentary for the design of the UHPC connection details. Embedment lengths (l_d), lap splice lengths (l_s), cover and spacing requirements for reinforcement using UHPC are provided in Table 1. AASHTO LRFD specifications for lap splice lengths of a Class B splice (100% of reinforcement spliced and A_s provided / A_s required < 2) are also shown in Table 1 for comparison. The values for the lap splices calculated by AASHTO procedures assume grade 60 non-epoxy coated reinforcement and a 22 ksi compressive strength for the UHPC in the joint. AASHTO criteria is limited to 15 ksi but was assumed to apply. These lengths are important for all the ODOT UHPC projects.

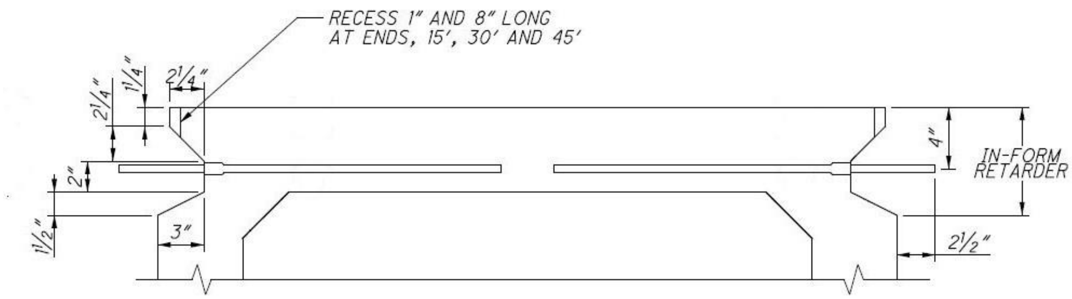
Table 1: UHPC and AASHTO Connection Design Recommendations

Bar Size	Criteria	cover (in.)	l_d (in.)	l_s (in.)	spacing (in.)
No. 4	UHPC	≥ 1.5	4.0	3.0	≤ 3.0
		≥ 1.0 but < 1.5	5.0	3.75	≤ 3.75
	AASHTO	≥ 2.5	20.0	25.96	≤ 6.0
No. 5	UHPC	≥ 1.875	5.0	3.75	≤ 3.75
		≥ 1.25 but < 1.875	6.25	4.69	≤ 4.69
	AASHTO	≥ 2.5	24.9	32.4	≤ 6.49
No. 6	UHPC	≥ 2.25	6.0	4.50	≤ 4.5
		≥ 1.5 but < 2.25	7.5	5.63	≤ 5.63
	AASHTO	≥ 2.5	29.9	38.9	≤ 7.78

Research Approach

Sollars Road Bridge:

The Sollars Road Bridge was constructed in the summer of 2014 in Fayette County. It was the first bridge in the U.S. to utilize UHPC and dowel bars in the longitudinal joints of an adjacent prestressed concrete box beam bridge. No transverse dowel bars were used, and beam diaphragms only existed at the ends. Figure 2(a) shows details of the longitudinal joint used in the bridge. The dowel bar system had two parts, where the first part was embedded in the beam 18 in. and contained a female threaded end. The part that was embedded in the shear key had a length of 4.75 in. and had a male threaded end allowing it to be screwed into the part embedded in the beam. Figure 2(b) shows the joint during bridge construction.



(a)

(b)

Figure 2: Sollars Road Longitudinal Joint (a) Detail (b) Field Installation

The bridge was heavily instrumented and monitored during construction, loading, and for approximately one year after being open to traffic. The County Engineer noted a longitudinal crack in the asphalt pavement in the late summer of 2017. The research team visited the site in August of 2017 to determine if original instrumentation was still usable and to visually inspect the bridge. All instrumentation was still usable and in working order. The longitudinal crack in the pavement was along the centerline of the pavement and along the pavement seam between lanes. Since the bridge consisted of seven adjacent box beams with the lanes centered on the bridge, it was determined this longitudinal crack was not reflective as it existed over the center of the middle beam and was caused by the paving operations. Figure 3 shows the longitudinal crack in the asphalt overlay. In addition, no leakage between beams has been observed.

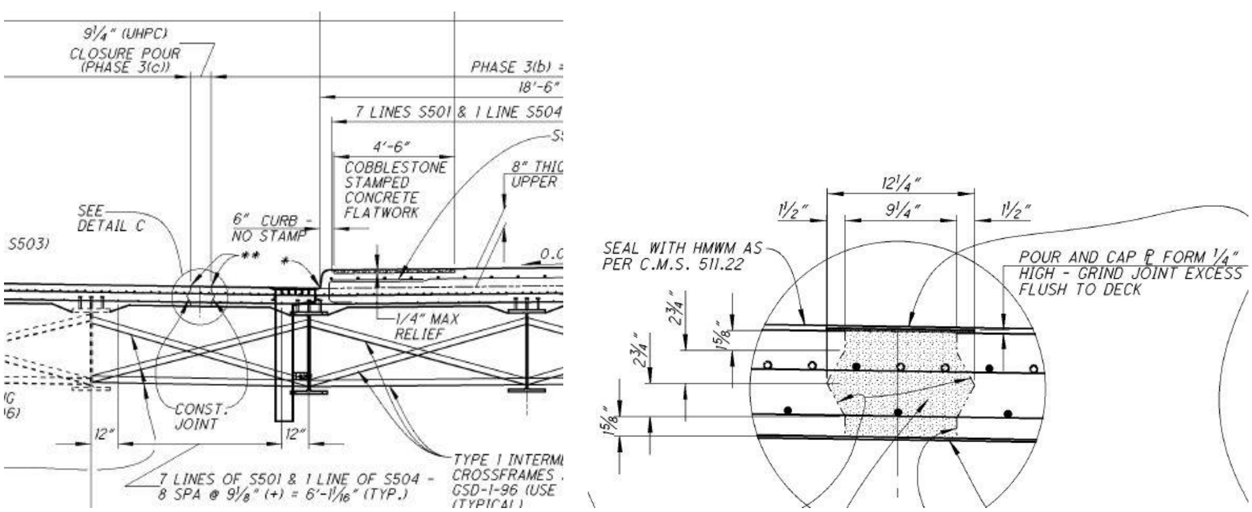
Figure 3: Sollars Road Bridge Asphalt Crack

The bridge was also load tested and instrumentation was monitored for behavior and performance in October of 2017. The truck load positioning matched truck loading performed in 2014 so results could be compared. The internal longitudinal strains near the top and bottom of Beams 1-3 at mid-span were nearly identical for the data collected in 2017 compared to 2014. This similarity in strains shows that the bridge is behaving in the same manner as it was prior to opening to traffic and implies no cracking of the shear keys. Additional data collected from other instrumentation at the quarter span in the beams, in the shear keys, and on the dowel bars in the shear keys also showed consistent results from 2017 as compared to 2014.

In addition, the instrumentation was monitored for daily thermal changes from early to late October of 2017. The data showed strain behavior similar to that noted previously. However, direct comparison was not possible due to differences in temperature from monitoring in July, August, and December of 2014 and January of 2015 to October of 2017. Additional information on the Sollars Road Bridge can be found in Appendix B.

LIC 310 Bridge Closure Pour:

LIC-310-0096 was a continuous four span bridge project over IR70. The project included multiple phases and widening of the existing bridge. Phase 3(c) of the project involved a 9¼" UHPC closure pour between a portion of the modification of the existing superstructure (Phase 3(a)) and an additional shared use pedestrian path (Phase 3(b)). Figure 4(a) shows a cross-section of a portion of the bridge including the closure pour. The higher dead load due to the addition of the sidewalk on the left side caused concerns related to the differential dead load deflection between the left and right sides of the closure pour. There were concerns this would generate stresses in the cross frames and deck without the usage of the closure pour. To alleviate concerns related to differential dead load deflection, the closure pour was located in the wheel path of the outside lane. The detail for the UHPC closure pour joint is provided in Figure 4(b). The joint had a single No. 5 longitudinal rebar in the top and bottom. Two additional No. 4 longitudinal bars existed in the top of the joint over piers. Transverse No. 5 bars were spaced at 5¾" in the top and bottom and were continuous through the joint. Additional information on this project can be found in Appendix C.



(a) (b)

Figure 4: LIC-310-0096 UHPC Closure Pour (a) Bridge Cross-section (b) Joint Detail

The location of the UHPC joint resulted in it being subjected to a variety of stresses from loading as well as daily and seasonal environmental changes. Since the deck was composite, the UHPC joint in the longitudinal direction was subjected to positive flexure within the span and negative flexure over the piers. In the transverse direction, the behavior of the deck subjected the joint to primarily positive flexure due to the joint being between girders.

A total of 15 strain gages were installed in the UHPC joint as well as the surrounding deck to monitor performance. Three strain gages were installed on June 21, 2017 in the driving portion of the deck before concrete placement. On July 28, 2017 instrumentation was installed in the UHPC joint. The details of the instrumentation can be found in Appendix C.

The UHPC was mixed and placed in the joint the evening of August 3, 2017. Figure 5(a) shows the overall process. The UHPC was moved from the mixers to the joints by wheelbarrows and placed into a wood chimney as shown in Figure 5(b). This allowed the UHPC to flow into the joint and form a hydraulic head to fill the joint. The UHPC was placed approximately ¼ in. higher than the surrounding road surface and then later ground flush. This is due to the initial shrinkage of UHPC during setting and to assure no low spots exist from the possibility of trapped air. The joint was also sealed on top with plywood to allow the UHPC to flow above the joint.

(a) **(b)**

Figure 5: LIC 310 UHPC Placement

Data was collected during UHPC placement and various periods throughout the construction and after opening of the bridge to traffic. Initially, the tensile strains were approximately equal and did not exceed 100 microstrain. Compressive strains exceeded 200 microstrain and the top of the UHPC and deck typically showed higher compressive strains than the bottom of the UHPC and deck. Temperatures measured by instrumentation in the top of the UHPC and deck were typically more extreme than the bottom of the UHPC and the deck. The higher high and lower low temperatures in the top also occurred slightly before the bottom. The top temperatures occasionally exceeded the ambient temperature measured near the surface of the deck. The highest measured tensile strains in the UHPC occurred longitudinally at the pier not long after placement but did not exceed 150 microstrain. Data was also acquired during truck loading of the bridge on September 9, 2017 prior to opening in order to have a known weight on the bridge. However, the data from the truck load showed minimal strain as the truck load relative to the bridge stiffness was minimal.

During inspection in April of 2018, small hairline cracking was noticed in the deck and the UHPC closure pour (see Figure 6). This cracking was not observed during inspections in the Fall of 2017.

Figure 6: UHPC Cracking

GAL 160 Precast Deck Panel System Joints:

ODOT District 10 planned to use UHPC for the joints between precast concrete deck panels and between the panels and the steel girders on a bridge in Gallia County on Route 160 (GAL 160-18.84). The three span continuous steel beam bridge had spans of 58 ft., 72.5 ft., and 58 ft. The deck panels were to be full width except near the bridge ends due to the high 35^{o} skew. The research team assisted in the project by reviewing the design, providing information obtained from other states that have used UHPC in this manner, and providing their own expertise and professional contacts related to the material. However, the Department ultimately directed to construct a traditional cast-in-place deck for reasons unrelated to UHPC. Therefore, UHPC was not part of this project. Further details of the original panel joint design can be found in Appendix D.

Research Findings and Conclusions

UHPC in Ohio is performing well to date. It has superior properties to conventional concrete and cementitious grouts. UHPC's unique properties make it a prime candidate for connections and areas where limited space is available, and bond, strength, and durability are highly necessary. Other states have utilized UHPC in modified girders, waffle deck panels, and overlays. The more recent usage of UHPC is in connections. UHPC has been primarily used in connections between precast concrete deck panels, the flanges in prestressed concrete decked bulb tees girders, the longitudinal connections between adjacent box beams, the continuity connection at piers in terms of link slabs, and connections between precast concrete elements such as pier caps/columns/footings. More recent unique proposed usage of UHPC includes UHPC forms filled with conventional concrete to create composite sections of structural members which contain UHPC in critical stress and degradation areas and conventional concrete in less critical locations.

Sollars Road Bridge:

The UHPC longitudinal joints in the adjacent prestressed concrete box beam bridge is performing well based on visual inspections and data obtained soon after opening the bridge to traffic in 2014 and more recently in late 2017. The design of the UHPC longitudinal joint included dowel bars but eliminated intermediate diaphragms, transverse post-tensioning, and a composite deck. This design may be a legitimate alternative to solve the issue of cracking in the longitudinal joints (shear keys) and associated reflective cracking in composite decks for adjacent prestressed concrete box beam bridges. This improved behavior may result in longer service life performance of these popular bridges.

LIC 310 Bridge Closure Pour:

The UHPC used for the closure pour in the location of a wheel path is performing well, even though micro-cracking has been observed. Strains from truck loading were minimal and measured tensile strains during various temperature changes did not exceed 150 microstrain. Therefore, the measured strains would not cause cracking.

GAL 160 Precast Deck Panel System Joints:

UHPC was proposed to be used in the GAL 160 precast concrete deck panel connections, but, the Department ultimately directed to construct a traditional cast-in-place deck for reasons unrelated to UHPC. The proposed end panels had a unique connection to deal with a large skew. The project would have been the largest placement of UHPC material in Ohio.

Recommendations for Implementation of Research Findings

Advantages

This material should continue to be explored for usage in Ohio's transportation infrastructure. UHPC has many advantages over conventional concrete or grout materials. It has a very high compressive strength typically exceeding 22 ksi. The compressive strength also increases at a rapid rate with 14 ksi typical in 2 to 3 days under normal ambient conditions. This rapid strength gain can be an advantage in accelerated bridge construction (ABC) projects where time is critical. The high compressive strength is accompanied by a high tensile strength near 1 ksi. In addition, the fiber content in UHPC gives it a post cracking strength that does not exist in conventional concrete or grout. If cracking occurs in UHPC, it is resisted by fibers and results in smaller cracks that are greatly dispersed compared conventional reinforced concrete and grouts. UHPC has a high bond strength which shortens development lengths of reinforcement and results in improved interface behavior between UHPC and substrate materials (conventional cast concrete or precast concrete) in connections or repairs. UHPC is also flowable which allows it to enter tighter spaces and around reinforcement.

Disadvantages

There are also disadvantages with UHPC. The cost of UHPC is very high relative to conventional concrete and grouts. Part of this higher cost is due to limited suppliers, but this is rapidly changing. The high cost is also the result of the material being new and unfamiliar to designers and contractors which often build in conservatism in designs and bids. The high cost is also a result of smaller quantities often being used in critical locations such as connections. However, the relative high cost of a small quantity is often a much smaller portion of the total project cost. In addition, improved long term performance may offset initial high costs. Currently design specifications from AASHTO do not include criteria related to UHPC. However, efforts are underway by numerous organizations and groups to improve on documentation to assist designers, owners, and contractors. Significant research by the Federal Highway Association and academia, and usage of UHPC by other states across the U.S. is rapidly increasing the needed knowledge and information to increase confident usage.

Recommendations

- Regardless of the UPHC usage, the owner and contractor should work closely with the UHPC supplier early in the project process and throughout the project so all involved can be well informed to assure success.
- UHPC is flowable so forms must be watertight.
- UHPC is placed in chimneys and allowed to flow into forms. The chimneys create hydraulic head pressure to assist in flow and filling the forms.
- If UHPC is to be bonded with previously cast concrete, bond at the interface surface between the UHPC and previously cast concrete is greatly improved if the surface has an exposed aggregate finish and is therefore highly recommended. This can be done with a form retarder and the surface power washed after form removal.
- The surface between UHPC and previously cast concrete should be prewetted before UHPC placement.
- Placement of UHPC at high ambient temperatures should be avoided or ice may be required during mixing.
- If the UHPC is intended to be a final surface, it should be cast high (approximately ¼ in.) and then ground flush. The rapid strength gain of UHPC (14 ksi in 2 to 3 days) and its final strength (22 ksi or more) makes grinding more difficult if done after an extended period after placement.

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Appendix A: Literature Review

UHPC - General

Ultra-High-Performance Concrete (UHPC) was developed commercially in France during the early 1990s. UHPC resulted from research into macro-defect-free (MDF) and densified with small particles (DSP) concretes from the 1980s. UHPC is effectively a new class of concrete with improved strength, tensile ductility, excellent bond characteristics, and superior durability from very low permeability. Unfortunately, the definition of UHPC varies widely based on the magnitude of properties. UHPC, also known as Ultra-high-performance fiber reinforced concrete (UHPFRC), was originally defined by the *Association Française de Génie Civil* (AFGC) as a material with a cement matrix, a compressive strength that exceeds 22 ksi, and containing steel fibers (AFGC, 2002). The steel fibers in UHPC create what appears to be ductile behavior under flexural tension from strain hardening and fiber pullout (Graybeal and Hartmann, 2003).

Different brands of UHPC are produced by several different cement companies. These different brands vary in mechanical properties and mix designs. Though several UHPC products are available commercially, one of the more prevalent UHPC mixes in North America is Ductal. This product is made and distributed by LaFargeHolcim North America and various formulations are available. The Ductal mix JS 1000 has been used in several joint bridge projects (Steinberg, et. al., 2015b; White, 2014). Ductal UHPC thixotropic mix formulation has been used for bridge deck overlays and remains fluid when agitated but stays in place when left to set. This allows placement on decks with up to 10% slope. A fast setting mix can also achieve 12 ksi compressive strength within 12 hours after placing.

UP-F2, UP-F3, and UP-F4 are UHPC mixes distributed by King Construction Products out of Burlington, Ontario, Canada. The product was developed by Polytechnique, Montreal, which is the engineering school at the University of Montreal. The difference in the three formulations is the amount of fibers in each mix. UP-F2, UP-F3, and UP-F4 have 2%, 3%, and 4% fibers by volume in the mixes, respectively. The 28-day compressive strength for all mixes is claimed to be 17.4 ksi (14.5 ksi for design). The tensile strengths increase with fiber content while all other material properties are not affected as much.

BFUP and BCV were UHPC products available from Béton Vicat. However, the company appears to have been acquired by LaFargeHolcim.

Ducorit by Densit is a pumpable UHPC that has been developed for grouted structural connections. Several different types of Ducorits have been developed using different aggregates such as quartz and bauxite. The different Ducorits include D4, S5, $S5_R$, S2 and S1. The differences in properties of the various Ducorits are shown in Table A1.

Table A1: Ducorit Properties (Densit, 2017)

Property	Ducorits D4	Ducorits S5	Ducorits $S5_R$	Ducorits S2	Ducorits S1
Compressive Strength (ksi)	29	18.85	18.85	17.5	17.5
Static Modulus of Elasticity (ksi)	10,000	8,000	8,000	6,800	5,000
Tensile Strength (ksi)	1.5	1.0	1.0	0.87	0.725
Flexural Strength (ksi)	3.4	2.6	2.6	1.45	2.0
Density (pcf)	171	152	149	147	140

COR-TUF is a UHPC that was developed and patented by the U.S. Army Corps of Engineers Engineer Research and Development Center (ERDC).

K-UHPC has been developed by the *Korean Institute for Civil Engineers and Building Technology* and was utilized in a bridge in Iowa (Kim, 2016).

Taktl is a UHPC product used in architectural panels out of Turtle Creek, PA (http://www.taktl-llc.com/).

In addition, research has been conducted to allow users to develop and mix non-propriety UHPC for use in bridge construction (Graybeal, 2013; Willie and Boisvert-Cotulio, 2013). The South Carolina Department of Transportation funded research to develop a UHPC mix from local materials (Rangaraju, et. al., 2014). A similar effort was also funded by the Washington Department of Transportation.

Table A2 shows a few representative mix designs for UHPC (Russell, 2013 and Rangaraju, et. al., 2014). As shown in Table A2, UHPC is made mostly from portland cement and sand. It should also be noted that large aggregate is not used in these mixtures.

UHPC is flowable when thoroughly mixed and requires sealing of joints to eliminate leakage. Figure A1 (Graybeal, 2010a) shows a procedure of placing UHPC directly into the joint. Another procedure for UHPC placement involves covering the top of the joints and then using "chimneys" placed along the joints that allow the flow of the UHPC along the joint. The usage of the chimneys is shown in Figure A2 (Steinberg, et. al., 2015a). The flow of the UHPC in joints has been studied to determine the effects of side surfaces of the joint region and reinforcing bars on the alignment of fibers (Walsh, et al., 2018). The fiber alignment can influence the properties in different directions, which can lead to stress concentrations.

Table A2: UHPC Mixes (lb/yd^3)

Component	Ductal®	COR-TUF*	SCDOT
Portland Cement	1,200	1,260	1,249
Silica Fume	390	490	250
Quartz Powder	355	349	-
Fine Sand	1,720	1,218	1,873
Steel Fibers	263	391	270
Superplasticizer	-	22	RQ
Water	184	262	300
HRWR	52	-	-
Accelerator	51	-	-

*Quantities estimated based on weight percentage.
RQ indicates required quantity to obtain 150% full flow.

Figure A1: UHPC Placement (Graybeal, 2010a)

Figure A2: UHPC Placement in Chimneys (Steinberg, et. al., 2015a)

UHPC Bond

Bond between reinforcement and the UHPC is an important aspect in design. The superior bond characteristic of UHPC has been studied in a number of research studies. Lubbers (2003) investigated the bond performance between UHPC and unstressed prestressing strands. In conventional concrete, the average load to embedment ratios were approximately 1.3 kips/in. These ratios increased to over 2 kips/in. for standard ½ in. diameter strand and up over 2.5 kips/in. for ½ in. oversized strands.

Graybeal (2015) investigated a lap splice development of unstressed strands embedded in UHPC. The authors used two different UHPC mixes one with steel fiber reinforcement and another with PVA fibers. The cross-section of the specimens was 3 in. by 5 in. with a cover of 1.25 in. for 0.5-in. strands and 1.2 in. for 0.6-in. strands. The results showed that the specimens with 0.5-in. strand, 16 in. lap length, and UHPC with steel fibers reached 98% of the nominal strand capacity (270 ksi). The specimens with 0.6-in. strand, 24 in. lap length, and UHPC with steel fibers reached to 104% of the nominal strand capacity (270 ksi). The specimens with 0.5- in. strand, 36 in. lap length, and UHPC with PVA fibers reached to 103% of the nominal strand capacity (270 ksi). Also, the results demonstrated that UHPC specimens with steel fibers were stiffer than UHPC with PVA fibers because tighter cracks in UHPC specimens with steel fibers closed at the conclusion of each test. Furthermore, the author showed that bond stress of the specimens with 0.5 in. strand and UHPC with steel fibers was 1.6 ksi over a 18-in. embedment length, that bond stress of specimens with 0.6-in. strand and UHPC with steel fibers was 1.3 ksi over a 24-in. embedment length, and that bond stress of specimens with 0.5-in. strand and UHPC with PVA fibers was 0.8 ksi over a 36-in. embedment length.

In addition of the importance of UHPC bond to reinforcement, the bond of UHPC to other concrete is important when considering joints in precast concrete systems. The adhesion and friction between the two materials, which are key parameters in determining the strength of the connection, depend on surface conditions. Hussein, et al., (2016) investigated the adhesion between UHPC and high-strength concrete for various surface conditions, and determined friction coefficients.

Laboratory Testing of UHPC Connections

Vitek, et al. (2016) investigated a UHPC joint between precast deck panels with straight and looped bars. The overall test sample sizes were approximately 9 by 2 ft and almost 10 in. thick. The UHPC joint between the panels was tested to simulate a longitudinal joint over steel girders. The results did not show reinforcement bond failures in the UHPC joint. For the looped reinforcement specimens, cracking occurred in the interface between UHPC joint and precast slab. Cracking in ordinary concrete occurred for the straight lapped bar specimens.

Graybeal (2010b) investigated test samples to emulate the performance of field-cast UHPC for connections between precast deck panels. This study used four specimens to simulate the transverse connections between full-depth precast deck panels, which have connections similar to DBT girders. The specimens differed in the reinforcement details within the joint: (1) headed black bars, (2) hairpin epoxy-coated bars, (3) galvanized straight bars, and (4) black straight bars. All the specimens had a female diamond-shaped shear key with a 6-in. width at the top and bottom. Figure A3 provides the section details for the headed bar. The specimens were loaded cyclically and statically. The cyclic loading test results showed that no cracks were formed along the interface between UHPC joints and precast panel. Since the interface did not crack, none of the four discrete reinforcement details was engaged and the best performing detail could not be established. The cyclic load testing revealed the cracking load of the specimens is greater than 16 kips and less than 21.3 kips. In addition, static testing resulted in concrete crushing above localized cracking after pullout tensile capacity of fibers in the UHPC and yielding of the panel reinforcement had occurred. Overall, the behavior of the precast panel with transverse connection exceeded the behavior of the monolithic deck.

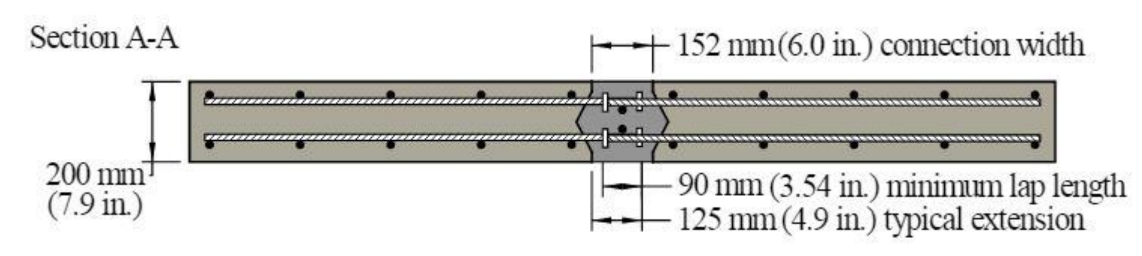

Figure A3: UHPC Deck Panel Detail (Graybeal, 2010b)

Lee and Lee (2015) investigated the flexural behavior of precast concrete connections filled with ultra-high-performance fiber-reinforced concrete. The surfaces of the precast members were vertical and did not include a shear key. In addition, the surface of the precast concrete did not appear to have any preparation such as an exposed aggregate finish. The research also included evaluating lap splices in UHPC beams. The results of the lap splice beams showed a distance of $10d_b$ was sufficient to transfer yield strength of the bars. However, the precast beam testing showed the lap splice length only needed to be more than $7d_b$ to develop the moment capacity of a monolithic concrete specimen.

Vitek, et al. (2016) investigated two types of steel-concrete composite beams. The control beam (Type 1) consisted of a cast in-situ slab without any joint and the second type of composite beam included a precast concrete slab with a UHPC joint over the top flange of beam. The results of testing showed cracks in the cast-in-place slab crossed the complete width of the slab, whereas cracks in the precast slab with the UHPC joint were primarily in the precast portion with a few of them passing through UHPC joint. Crack widths were also small (0.008-0.012 in.) in both systems. In addition, longitudinal cracking was in the middle of in-situ slab above shear connectors. In the precast slab, the longitudinal cracking was in ordinary precast concrete slab or in the interface between UHPC joint and precast slab. The composite beam behavior for the UHPC system also showed higher stiffness at higher loads and an increase in load carrying capacity.

In addition to laboratory testing transverse connections to simulate deck panel connections, Graybeal (2010b) also tested connections in the laboratory to simulate longitudinal connections of DBTs. The differences in support conditions and loading pattern are shown in Figure A4 (Graybeal, 2010c). The study used two specimens to simulate the longitudinal connections between deck bulb tee girders. The specimens differed in the reinforcement details within the joint which included headed black bars and black straight bars. All specimens had a female diamond-shaped shear key with 6-in. width at the top and bottom. Figure A5 provides the section details for the headed bars. The specimens were loaded cyclically and statically. For the headed bar specimen, cyclic loading was done for over 2 million cycles from 2 to 16 kips followed by almost 7 million cycles from 2 to 21.3 kips. During the cyclic loading to 16 kips no additional cracking was observed from cracking that was identified prior to testing. Cracking of the specimen after completion of cyclic testing is shown in Figure A6 (Graybeal, 2010b). Static testing of the headed bar specimen achieved a maximum load of 116.8 kips. Failure was precipitated by a punching shear failure of the precast deck panel near the loading area. The straight bar specimen was cyclically loaded differently. Over 57,000 cycles were applied from 2 to 16 kips, followed by an accidental overload of 70 kips. Cyclic loading was then continued for 10 million cycles from 3 to 21.3 kips, followed by over 1 million cycles from 3 to 32 kips and finally over 300,000 cycles from 3 to 40 kips. Cyclic loading was terminated as the majority of reinforcing bars fractured at the interface on the north side of the connection.

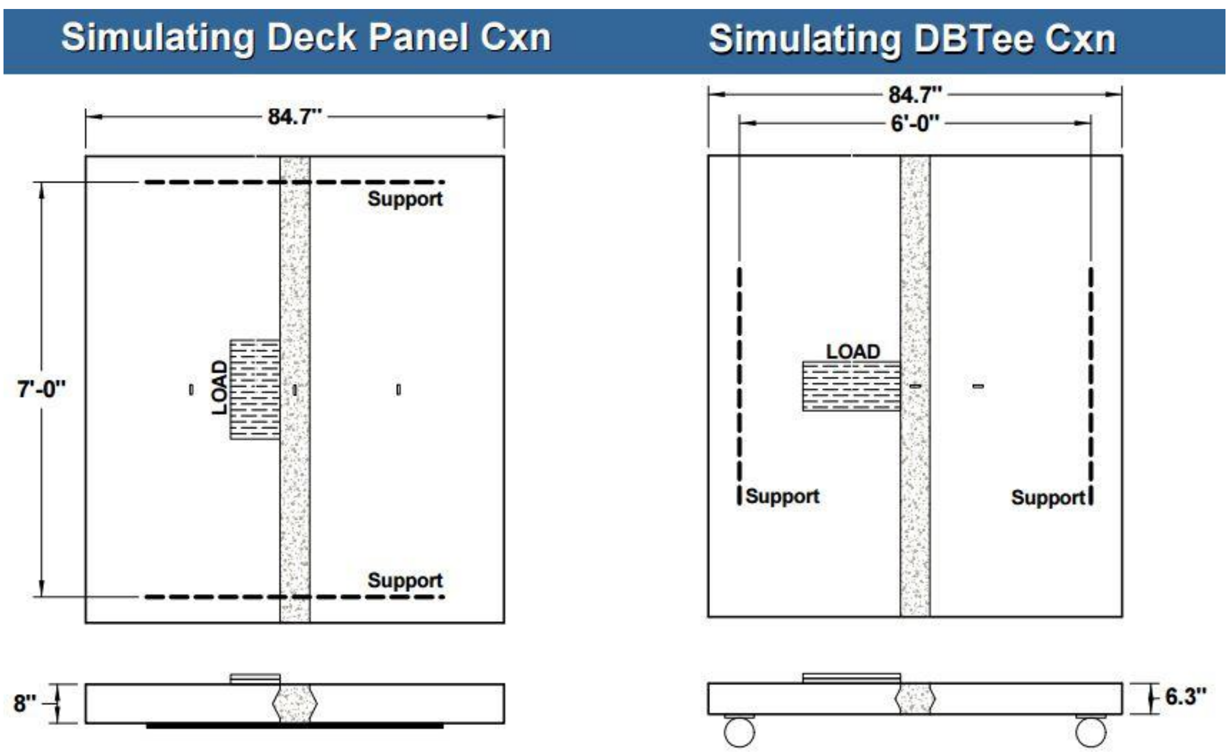

Figure A4: UHPC Testing Details (Graybeal, 2010c)

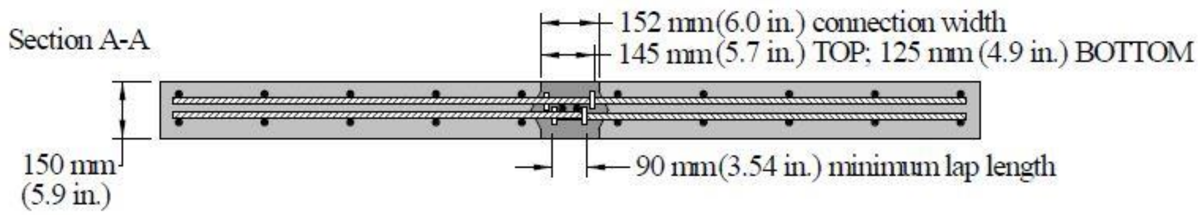

Figure A5: UHPC DBT Connection Detail (Graybeal, 2010b)

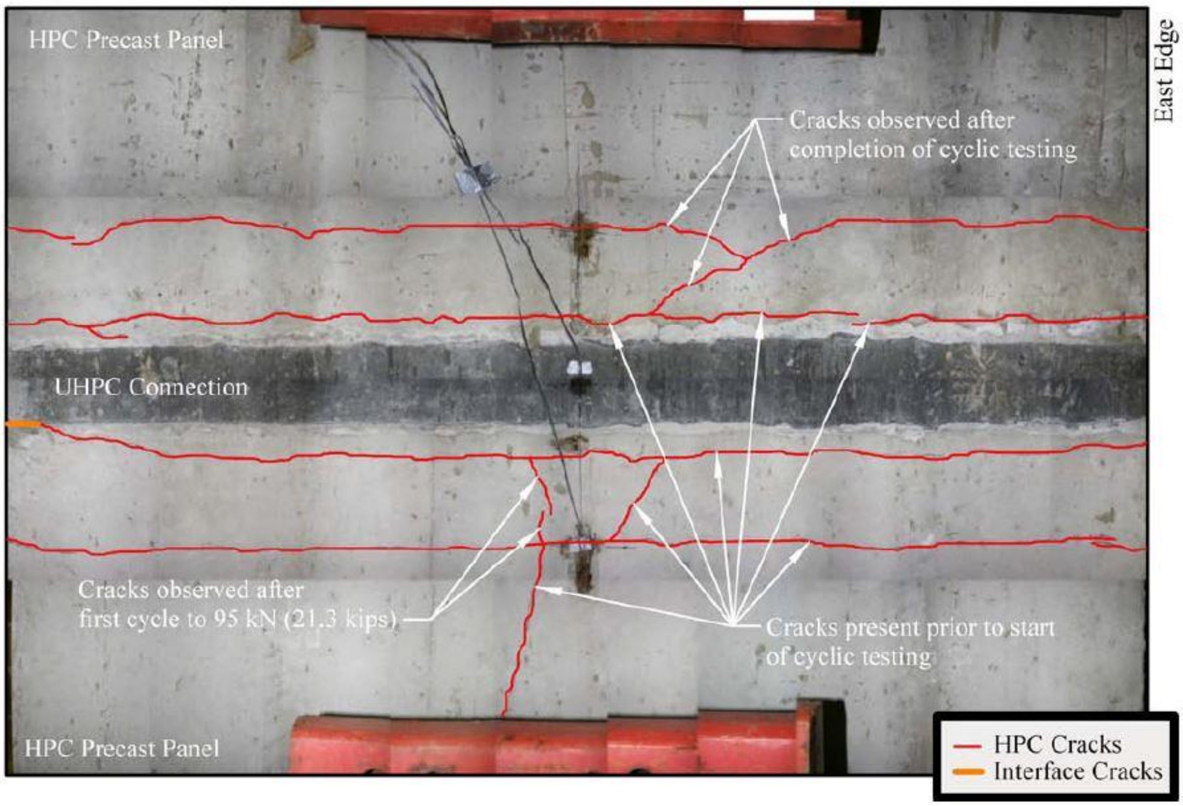

Figure A6: UHPC DBT Headed Bar Specimen Cracking after Cyclic Loading (Graybeal, 2010b)

UHPC Connection Field Testing

Though there have been a multitude of projects utilizing UHPC, there is a very limited amount of data collected on field performance. Overall bridge connections have been monitored; however, little information is available about the performance of UHPC field connections. In addition to instrumentations installed on the bridge girders (Steinberg, et al., 2015b), the UHPC joint of the Sollars Road Bridge in Fayette County, Ohio, contained instrumentation. The UHPC connection was monitored during early age behavior by Steinberg and his research team (Semendary, et al., 2017a). This particular research investigated the connection behavior as the UHPC gained strength, and the importance of the dowel bars during this period. Moreover, the effects of daily thermal changes on longitudinal and transverse behavior of the joint was monitored. It was shown that transverse strains became more compressive as temperatures increased and the compressive strains were reduced under dropping temperatures. This trend was similar to the behavior of the beams but the compressive strains in the shear key continued to increase over time. Reinforcing dowel bars in the joint also showed higher strains than the portions of the bars embedded in the beams. The dowel bars in the joint also showed higher changes in strain at early age for a given temperature change when compared to a similar temperature change a month or year later. Strains in the UHPC joint were also compared to strains measured in a non-shrink grout. Although strains in the non-shrink grout led to cracking, the strains in the UHPC did not lead to cracking. Semendary, et al., (2017b) also analyzed the results from the static loading of a bridge containing UHPC shear keys. This work has enabled the calculation of moment distribution factors for the bridge containing UHPC shear key connections (Semendary, et al., 2017c).

UHPC Transportation Uses

The first use of UHPC for a bridge project in the U.S. was the Wapello County Bridge in Iowa. Constructed in 2005, the bridge consisted of a conventional concrete deck placed on top of three 110-ft long modified Iowa bulb tee sections made with UHPC. The 45-in. deep bulb tees were modified by reducing the web thickness from 6.5 in. to 4.5 in., the bottom flange from 7.5 in. to 5.5 in., and the top flange from 3.75 in. to 2.75 in. (Endicott, 2007). Other unique sections have also utilized UHPC such as the Pi-Girder used in the 136th Street at Jakway Park Bridge in Buchanan County, Iowa, in 2008. This unique shape was specifically developed for the enhanced properties of UHPC and is shown in Figure A7 (Russell and Graybeal, 2013). Waffle deck panels have also been used in the Little Cedar Creek Bridge in Wapello County, Iowa, as shown in Figure A8 (Moore, 2012). The panels were 15 ft by 8 ft by 8 in. deep, and were installed on conventional prestressed concrete beams. Longitudinal and transverse joints between panels were also field cast with UHPC as shown in Figure A9 (Heimann, 2013). In addition, a fair amount of analytical work has been performed on the UHPC waffle deck panel systems (Garcia, 2007; Aaleti, et al., 2013).

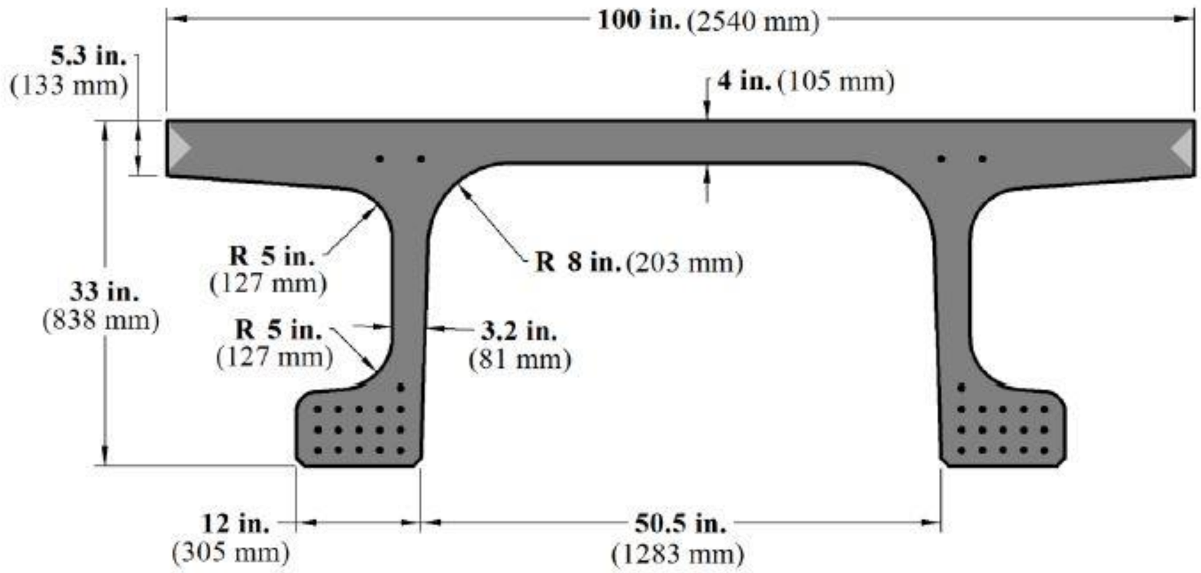

Figure A7: Jakway Park Bridge Pi-Girder (Russell and Graybeal, 2013)

Figure A8: Waffle Deck Panels of Little Cedar Creek Bridge (Moore, 2012)

Figure A9: Waffle Deck Panels with UHPC Joints (Heimann, 2013)

However, the cost of UHPC and the difficulty of modifying forms in order to produce non-standard shapes that fully utilize the properties of UHPC has led to its usage in smaller more critical applications. These applications include a variety of bridge joint connections that have limited space available and require a quality product to assure proper load transfer and durability. UHPC has been used in DBT bridges in the states of Idaho, Massachusetts, New York, New Jersey, and Oregon. Some of these bridges are shown in Table A3. The usage of UHPC in the longitudinal connection between DBT girders has been considered because of the superior properties of UHPC and the limited space available in the typical DBT joint.

Table A3: Constructed DBT Bridges with UHPC Longitudinal Joints

Bridge Name or Route	Feature	Location	Year
SR 31 (Forgham Street)	Canandaigua Outlet	Lyons, NY	2009
Fingerboard Road	Staten Island Expressway	Staten Island, NY	2011
SR 248	Bennetts Creek	Greenwood, NY	2011
SR 10 (Northhampton Street)	Manhan River	Easthampton, MA	2013
SR 46	Musconetcong River	Hackettstown, NJ	2014
SH 97	I-90 Overpass	Coeur d'Alene, ID	2016
US 30	Chenoweth Creek	Wasco County, OR	2017

The SR 31 bridge constructed in 2009 in Lyons, NY, consisted of eight deck bulb-tee beams with UHPC longitudinal joints. The joints consisted of straight epoxy-coated bars that projected from the beam flanges and were staggered as shown above the diaphragm reinforcement in Figure A10 (Shutt, 2009). The bars extended 4 in. or 6 in. from the edge of the girders which were only 6 in. thick at the flange edge. Kunin and White (2009) instrumented the beams of the bridge in Lyons, NY. The beams were modified *Prestressed Concrete Committee for Economic Fabrication* (PCEF) bulb tee girders by increasing the top flange thickness in order to eliminate the need for a separate concrete deck slab. The bridge had a span of 85 ft and was 42.75 ft wide with a 15-degree skew. Camber adjustment was performed prior to UHPC placement to provide a flat bridge deck as shown in Figure A11 (Royce, 2011). Once the UHPC obtained a strength of 14 ksi, the camber adjustment equipment was removed. It was determined the camber adjustment created a maximum locked-in compressive stress of approximately 400 psi and a maximum locked-in tensile stress of approximately 230 psi (Kunin and White, 2009). Load transfer across the joints and load distribution appeared to be performing well based on field load tests.

Figure A10: Lyons, NY DBT Bridge from below before UHPC Placement (Shutt, 2009)

Figure A11: Lyons, NY, DBT Bridge Leveling Operation (Royce, 2011)

Fingerboard Road over Staten Island Expressway I-279 consisted of two spans, each 103 ft. The bridge utilized 14 DBT girders per span. The girders were 49 in. deep with the top flange 6 in. deep. The longitudinal joints were 6 in. wide and bars extended straight from the beams for a distance of no more than 5½ in. Bars were spaced at 6 in. along beams and staggered between beams to result in a 3-in. spacing between bars in the joint. The bars were No. 6 in the top layer and No. 4 in the bottom layer. Both bars had 1 in. cover. Details of the joint can be seen in Figure A12. The bridge had a skew of 10°12'44" (10.212°). However, the beam ends were not skewed. Steel diaphragms were used at third points for each span.

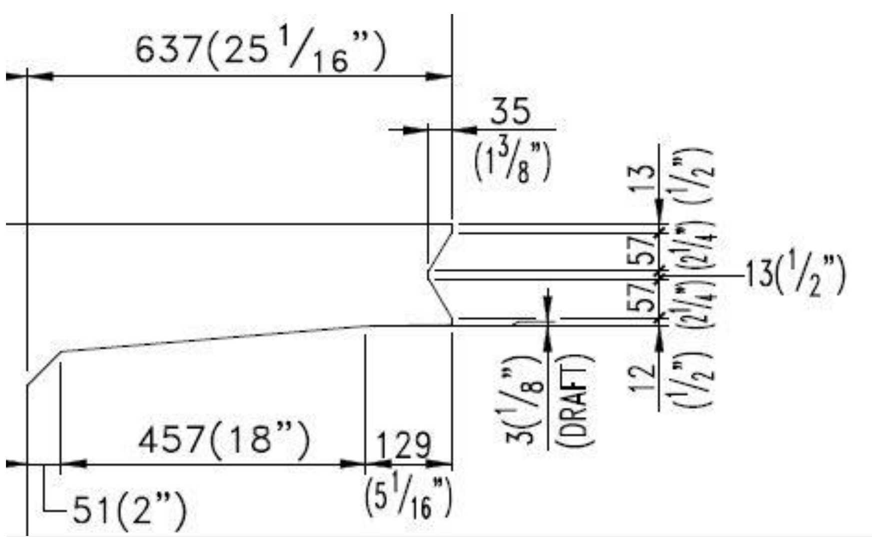

Figure A12: Fingerboard Bridge UHPC DBT Connection Detail (Northeast Prestressed Products, LLC, 2011)

The SR 248 bridge was constructed over Bennetts Creek in 2011 in Greenwood, NY. The bridge had a centerline bearing to bearing span of 134.5 ft and was 44.9 ft wide with a 35-degree skew. Diaphragms were used at the ends and near the quarter points. A total of nine deck bulb-tee beams with UHPC longitudinal joints created the superstructure. The beams were 55 in. deep with flanges that were 7 in. deep at the edges. The beam flanges were 4.5 ft wide. Longitudinal joints consisted of straight No. 5 bars that projected from the flange edges approximately 5.2 in. The bar spacing was 6 in. for each beam and was staggered to create a 3 in. spacing within the joint. A detail of the joint is shown in Figure A13. Sleeved bolts were used to secure forms for the top and bottom of the joint. It is interesting to note that after the UHPC joint hardened and the forms were removed, the sleeves within the joint were filled non-shrink grout. A concrete overlay with a nominal thickness of 2 in. was also placed on the bridge deck.

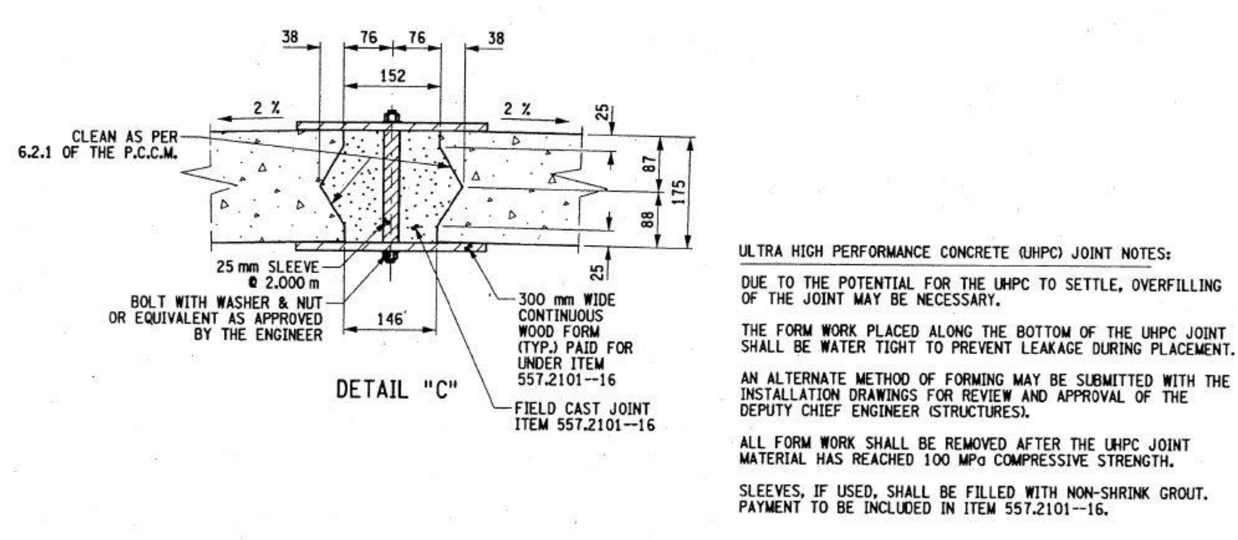

Figure A13: SR 248 Joint Detail Greenwood, NY (New York State Department of Transportation, 2010)

The SR 10 bridge in Easthampton, MA consisted of eight DBT sections that replaced a deficient bridge that spanned approximately 95 ft (White, 2014). The joints between the beams were 6 in. with the exception of the middle joint which was 8 in. The closure pours for the joints were done with UHPC and included looped No. 4 bars spaced at 6 in. as shown in Figure A14. The UHPC was placed ¼ in. higher than the edges of the DBT flanges by attaching plywood to the tops of the flanges. The UHPC was then ground flush with the girder surfaces. The entire deck was covered with a membrane waterproofing, a 1½ in. Superpave bridge protective course, and a 1½ in. Superpave bridge surface course. Cost estimates showed that the precast concrete DBT girders with UHPC was more economical than the alternatives of steel plate girders with a composite cast-in-place deck slab and precast New England bulb tees with a composite cast-in-place deck slab (White, 2014).

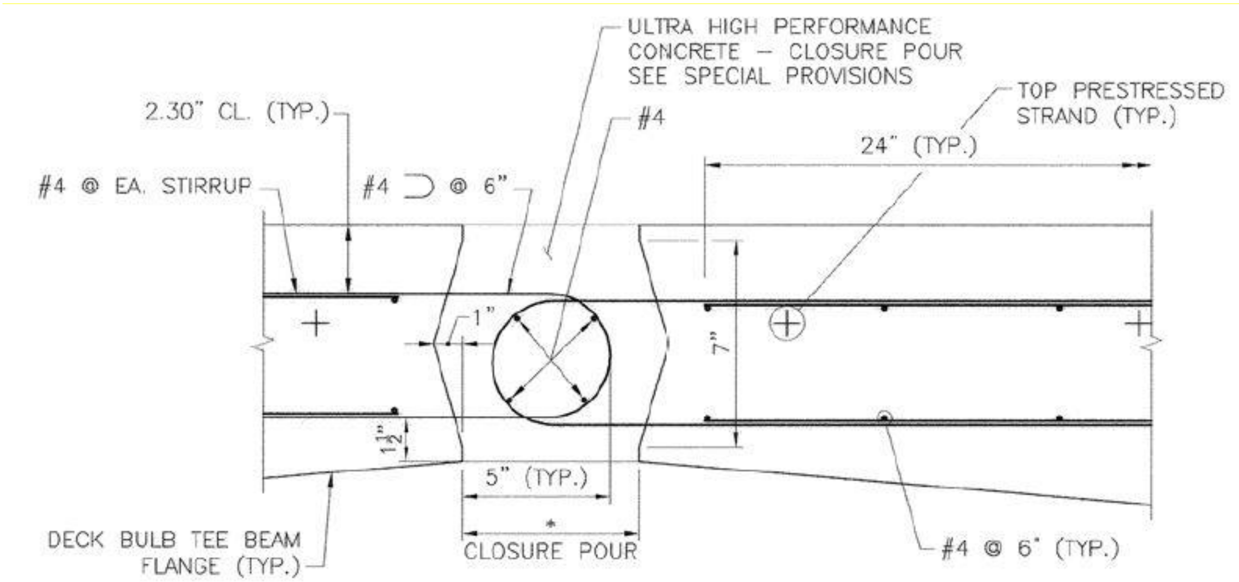

Figure A14: Easthampton, MA UHPC DBT Connection Detail (White, 2014)

The SH 97 bridge in Coeur d'Alene, ID had two spans, one 99 ft and the other 106 ft. Each span consisted of 6 DBT sections that were no more than 54.875 in. deep. The flange thickness varied along the member length from a maximum of 11.875 in. at the ends to a minimum of 8.5 in. near midspan for the longer span. The longitudinal joints between the flanges of the DBT's were 6 in. and contained straight No. 5 bars near the top and bottom of the flange (see Figure A15). The No. 5 bars extended 5 in. from the edge of the flange and had a spacing of 6 in.

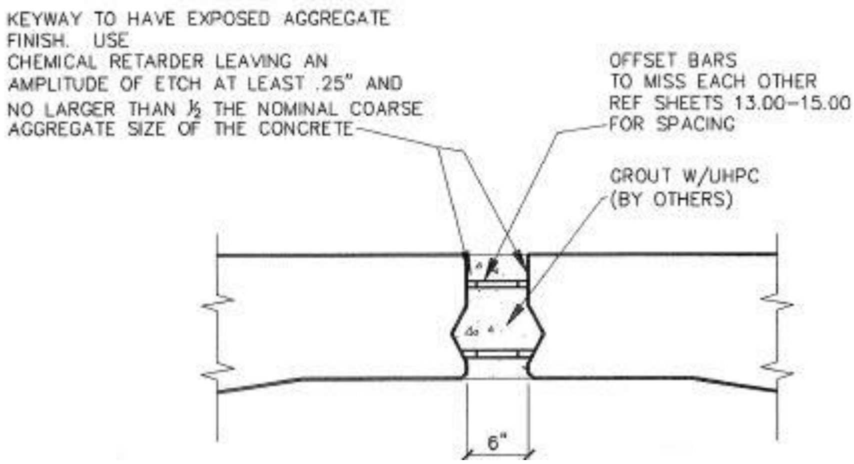

Figure A15: Coeur d'Alene, ID, UHPC DBT Connection Detail (Oldcastle Precast, Inc. 2016)

The US 30 bridge near The Dalles, OR is a single 87 ft span bridge. A total of five 45 in. deep modified DBT girders were used. The flange thickness varied from 6 in. to 6-1/16 in. for the exterior girders and from 6-1/16 in. to 6-3/16 in. for the interior girders. The flanges were sloped 2% to create the cross slope. The longitudinal joints between the flanges of the DBT girders were 8 in. wide and reinforcement extended 7 in. from the edge of the flange to allow for a 6 in. noncontact lap splice. No. 6 bars placed near the top of the flange had a 5 in. spacing while No. 4 bars in the bottom of the flange had a 10 in. spacing (see Figure A16).

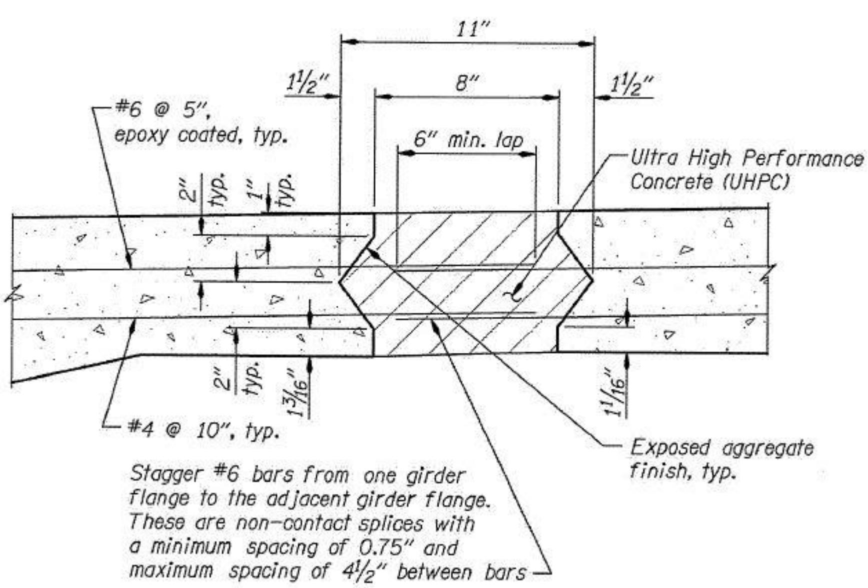

Figure A16: US 30 near The Dalles, OR, UHPC DBT Connection (Knife River, 2017)

The connections between deck panels have been used by the states of Delaware, Georgia, Illinois, Iowa, Maine, Minnesota, Montana, Nebraska, New York, Oregon, Pennsylvania, Utah, Vermont, and Washington as shown in Table A4 (Graybeal, 2014a). The panels have been placed on steel and prestressed concrete bridge girders.

Table A4: Deck Panel Bridges with UHPC Joints

Bridge Name and/or Route	Crossing Feature	Location	Year	Owner
SR 23	Otego Creek	Oneonta, NY	2009	NYSDOT
Seven Lakes Drive	Ramapo River	Sloatsburg, NY	2011	NYSDOT
US Route 30	Burnt River and Union Pacific Railroad	Huntington, OR	2011	ODOT
Dahlonega Road	Little Cedar Creek	Ottumwa, IA	2011	IowaDOT
I-481 Northbound	Kirkville Road	Syracuse, NY	2012	NYSDOT
I-690 Westbound	Peat Street	Syracuse, NY	2012	NYSDOT
I-690 Eastbound	Peat Street	Syracuse, NY	2012	NYSDOT
I-690 Westbound	Crouse Avenue	Syracuse, NY	2012	NYSDOT
I-690 Eastbound	Crouse Avenue	Syracuse, NY	2012	NYSDOT
SR 31	Putnam Brook	Weedsport, NY	2012	NYSDOT
SR 42 [South Bridge]	West Kill	Lexington, NY	2012	NYSDOT
SR 42 [North Bridge]	West Kill	Lexington, NY	2012	NYSDOT
US Route 87	BNSF Railroad	Moccasin, MT	2012	MDT
SR 10	Webster BrookÂ	Delhi, NY	2013	NYSDOT
SR 12	Spring Brook	Greene, NY	2013	NYSDOT
SR 38	SR 38	Newark Valley, NY	2013	NYSDOT
SR 907W (Hutchinson River Parkway)	US Route 1	Pelham, NY	2013	NYSDOT
I-690 Westbound	N. Salina Street	Syracuse, NY	2013	NYSDOT
I-690 Westbound	Onondaga Creek	Syracuse, NY	2013	NYSDOT
I-690 Eastbound	Onondaga Creek	Syracuse, NY	2013	NYSDOT

I-81 SouthboundÂ	E. Calthrop Avenue	Syracuse, NY	2013	NYSDOT
I-81 NorthboundÂ	E. Calthrop Avenue	Syracuse, NY	2013	NYSDOT
I-81 SouthboundÂ	E. Castle Street	Syracuse, NY	2013	NYSDOT
I-81 NorthboundÂ	E. Castle Street	Syracuse, NY	2013	NYSDOT
SR 1004	Cove Creek	Everett, PA	2013	PennDOT
SR 962G (Halstead Avenue)	US Route 17	Owego, NY	2013	NYSDOT
300th Street	Unnamed Creek	Primrose, NE	2014	Boone County
Renton North Bridge	Boeing Factory	Renton, WA	2014	Boeing
US 6	D&RGW Railroad	Spanish Fork, UT	2014	UDOT
Hooper Road	E. Main Street	Union, NY	2014	Broome County
CR 47	Trout Brook	Stockholm, NY	2015	NYSDOT
I-84 Westbound	Neversink River	Port Jervis, NY	2015	NYSDOT
I-84 Eastbound	Neversink River	Port Jervis, NY	2015	NYSDOT
I-87 Southbound	Shaker Road	Albany, NY	2015	NYSDOT
I-87 Northbound	Shaker Road	Albany, NY	2015	NYSDOT
North Court Street	over railroad	Lenox, NY	2015	NYSDOT
S. Peoria Street	I-290	Chicago, IL	2015	IDOT
E Franklin Avenue	Mississippi River	Minneapolis, MN	2016	MnDOT
I-81	SR-80	Tully, NY	2016	NYSDOT
Midway Road (SR 4041)	I-78	Bethel, PA	2016	PennDOT
PA-182 (Indian Rock Dam Road)	Codorus Creek Tributary	York, PA	2016	PennDOT
Power Dr	I-78	Strausstown, PA	2016	PennDOT
Snow Hill Rd	Stony Run	Cresco, PA	2016	PennDOT
SR-136	CSX RR	Eighty Four, PA	2016	PennDOT
SR-419 (Four Point Rd)	I-78	Schubert, PA	2016	PennDOT
Rte 196 (Maple St)	Glen Falls Feeder Canal	Hudson Falls, NY	2016	NYSDOT

Rte 960H (Mill St) (slatehill?)	Catatonk Creek	Candor, NY	2016	NYSDOT
I-81	NY 990G	Kirkwood, NY	2016	NYSDOT
SR-97 (Bridge 1)	Pea Brook	Long Eddy, NY	2016	NYSDOT
SR-97 (Bridge 2)	Pea Brook	Long Eddy, NY	2016	NYSDOT
SR-97 (Bridge 3)	Pea Brook	Long Eddy, NY	2016	NYSDOT
SR-211	Beech Creek	Athens, GA	2016	GDOT
SR-863 (Golden Key Road)	I-78	Allentown, PA	2016	PennDOT
VT-100	Mad River	Waitsfield, VT	2016	VTrans
SR-139 (Western Avenue)	I-95	Fairfield, ME	2016	MaineDOT
I-95 Northbound	SR1 / SR7	Newark, DE	2016	DelDOT

The first bridge in Illinois which contained deck panels with UHPC joints was constructed in 2015. The UHPC design was chosen over two other options that included internal post-tensioning design and a jacking with external post-tensioning system (Liu and Schiff, 2016). The bridge included a single longitudinal (see Figure A17) and multiple transverse UHPC joints (see Figure A18) between the half bridge width panels.

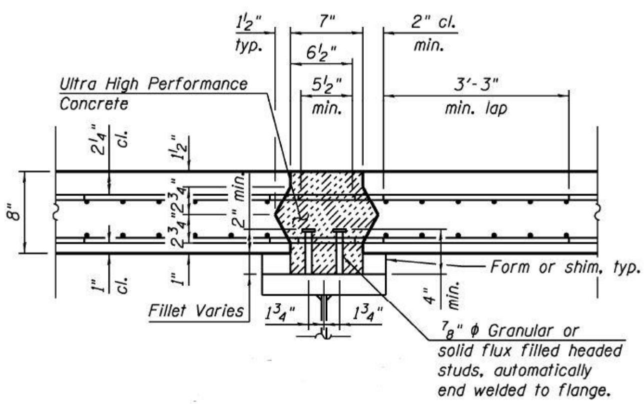

Figure A17: Illinois Longitudinal UHPC Deck Panel Joint (Liu and Schiff, 2016)

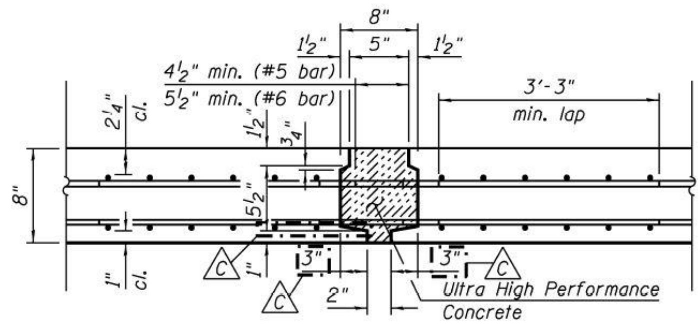

Figure A18: Illinois Transverse UHPC Deck Panel Joint (Liu and Schiff, 2016)

New York has used UHPC extensively in the joints of deck panels. Figure A19 displays the transverse UHPC deck panel joint used in the Pulaski Skyway bridge project (McDonagh and Foden, 2016). Figure A20 shows the haunches and shear pockets. Though these connections were not originally designed to use UHPC, the contractor opted to use UHPC in order to create a single UHPC pour for the transverse joints, haunches and shear pockets.

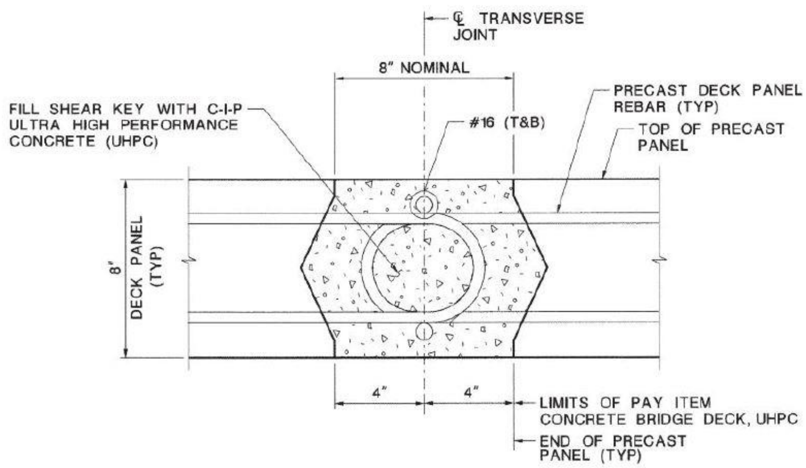

Figure A19: Pulaski Skyway Transverse UHPC Deck Panel Joint (McDonagh and Foden, 2016)

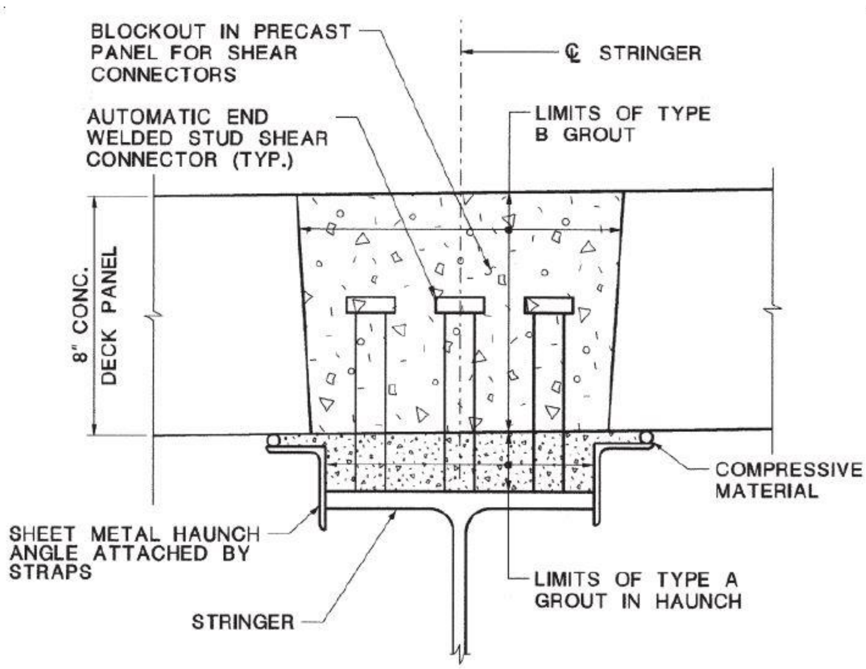

Figure A20: Pulaski Skyway Haunch and Shear Pocket for UHPC Deck Panels (McDonagh and Foden, 2016)

States are designing continuity in the decks over the piers while still designing the girders as simple spans. This is often being done with link slabs. Research related to link slabs has been performed by several researchers (Caner and Zia, 1998; Kim and Li, 2004; Kim, et. al., 2004; Lepech and Li, 2009; and Hajilar, et. al., 2017), but Larusson (2013) focused his research on UHPC link slab behavior. An example of a UHPC link slab used by the New York State is shown in Figure A21 (Royce, 2016).

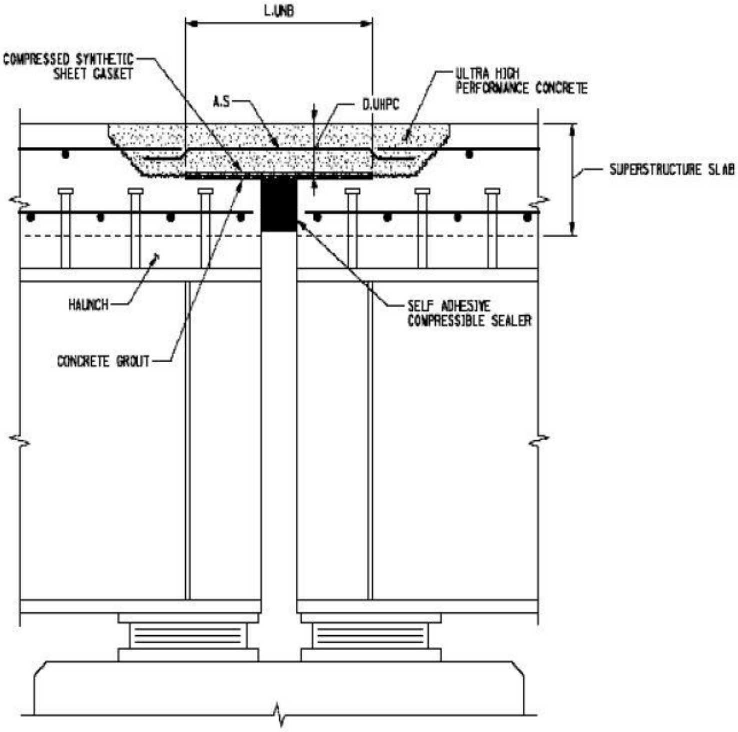

Figure A21: New York State UHPC Link Slab (Royce, 2016)

Appendix B: Sollars Road Bridge

Instrumentation

The box beams for the Sollars Road Bridge were fabricated in Kalamazoo, Michigan in May 2014 in a precast and prestressed concrete manufacturing facility. The typical box beam form was used, except the shear key shape was modified using wood. The wood form for the new shear keys can be seen in Figure B1. The form was coated with a retarder and the embedded ends of the dowel bar assemblies (with the female threaded ends) were placed on the red plastic tabs. Figure B2 shows the final installation. Figure B3 shows the shear key upon removal of the box beam from the forms and after power washing. The end result was a rough shear key surface with exposed aggregate that enhanced the bond between the beams and UHPC.

Figure B1: Shear Key Form

Figure B2: Beam Dowel Parts in Place

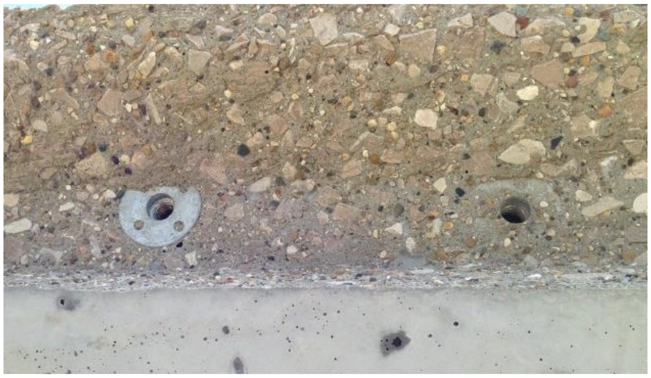

Figure B3: Power Washed Shear Key

The first three box beams were instrumented with vibrating wire strain gages embedded in the beams, and on the dowel bars. Five strain gages were used in each beam to monitor the strain in the longitudinal and transverse directions. Two vibrating wire strain gages, one in the top flange and one in the bottom flange, were placed longitudinally at the quarter span. Three vibrating wire strain gages, one longitudinal and one transverse in the top flange, and one longitudinal in bottom flange, were used at mid-span. The bottom gages were positioned between strands and the top gages mounted between the shear reinforcement. Figure B4 shows a longitudinal vibrating wire strain gage positioned in the form between the strands at the bottom flange. Figure B5 shows the gages on the top flange.

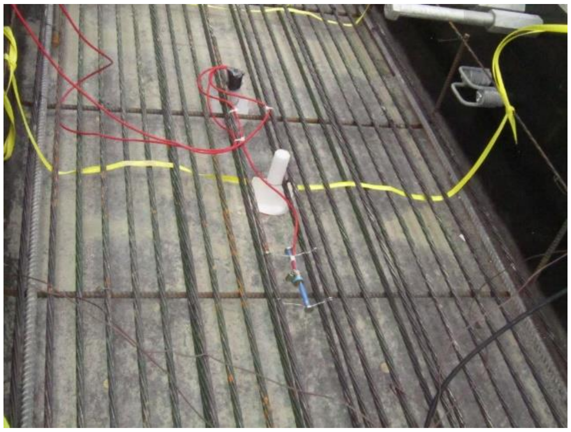

Figure B4: Vibrating Wire Strain Gage

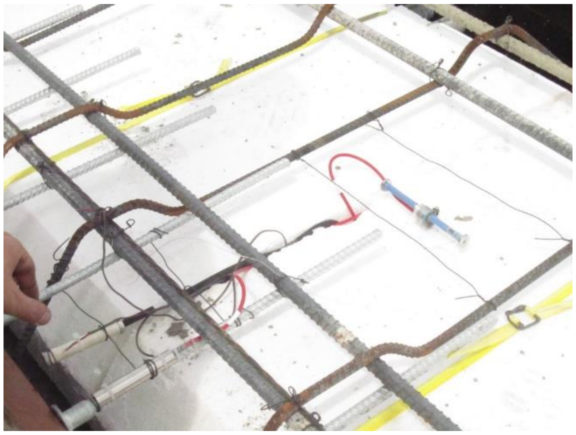

Figure B5: Instrumentation in Top Flange

The embedded ends of the dowel bars in each beam were instrumented using vibrating wire strain gages (see Figure B6), with one at the quarter span and one at mid-span. The gages were installed at a distance of 51 mm (2 in) from the threaded end. Beams 1, 2, and 3 had the instrumented dowel bars on right side of the cross section. Beam 3 was also instrumented with four thermocouples throughout the depth to measure the temperature along the depth of the beam (see Figure B7).

Figure B6: Instrumented Dowel Bar

Figure B7: Thermal Couples

On Saturday July 12, 2014, the box beams were transported to the site. Six vibrating wire strain gages were installed 38 mm (1.5 in) from threaded end on six dowel bar inserts. Instrumented dowel bars were installed to the left side of Beams 2, 3 and 4. Two instrumented dowel bars were used in each beam, one at the quarter span and one at mid-span (see Figure B8). For reference, the beams were numbered 1 to 7, from left to right, while facing the forward abutment.

Figure B8: Installed Instrumented Dowel Bars

On July 16, 2014, the three shear keys between Beams 1 - 4 were instrument with vibrating wire strain gages. Each shear key was instrumented with one transverse gage at the quarter span and one transverse strain gage at mid-span (see Figure B9). Shear keys 1 and 3 were instrumented with one gage at the quarter span and one gage at mid-span in the longitudinal direction (see Figure B10). After installation, the excess expandable filler material between beams was removed and the joints were covered with plywood, except for larger openings at the quarter points along the shear key.

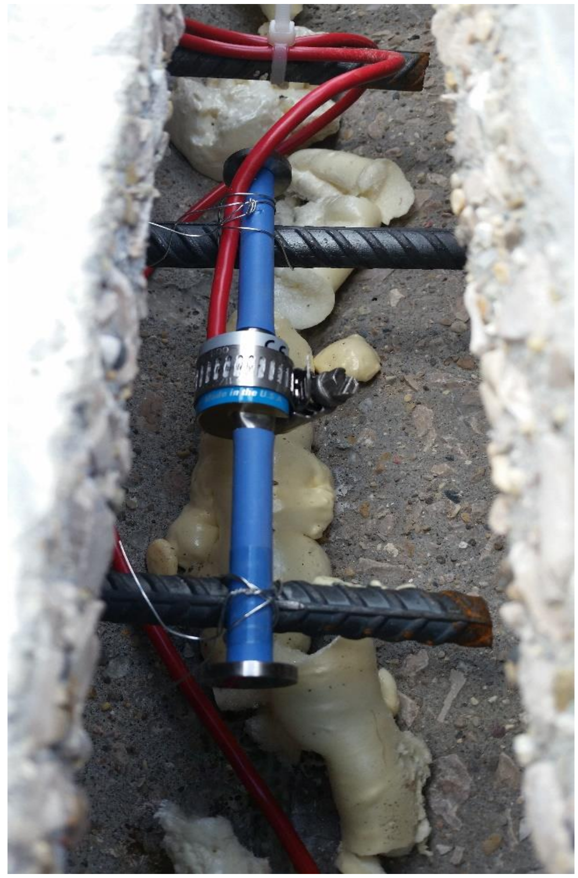

Figure B9: Transverse Shear Key Gage **Figure B10: Longitudinal Shear Key Gage**

On July 17, the shear key joints were cast using ultra-high performance concrete (UHPC). Two mixers were used to properly mix the UHPC. The UHPC was moved to the joints in wheelbarrows and placed into chimneys made of plastic buckets located at the larger joint openings (see Figure B11). The UHPC flowed into the joints, and the filling of the joints was assured by the hydraulic head of the UHPC in the chimneys. Instrumentation was connected to data acquisition systems in order to monitor the bridge as the UHPC cured. On July 22, the plywood forms were removed from the joints. No cracks were observed from inspection of the shear keys. On July 24, a waterproofing membrane was installed on the top of the bridge. The bridge was paved with an asphalt wearing surface on August 5. The following day, frames were set up underneath the bridge at the quarter span and mid-span (see Figure B12). On August 7, a total of 16 strain gages, seven LVDT's, and three thermocouples were installed to monitor the bridge.

Figure B11: UHPC Placement **Figure B12: Instrumentation Frames**

Bridge Testing

One August 8, 2014, two trucks were used to load test the bridge. The weights of the trucks were 249.5 kN (56.1 kip) and 237.5 kN (53.4 kip). Four static load configurations were used in the tests, and the trucks were positioned to obtain the maximum moment at mid-span. These load configurations were:

1. A single 56.1 kip truck load placed in the left lane
2. A single 53.4 kip truck load placed in the right lane
3. Two trucks placed side-by-side with a 109.6 kip total load
4. Two trucks placed back to back in the left lane for a 109.6 kip total load (see Figure B13).

Figure B13: Truck Loading

Test Results

The bridge was also load tested and instrumentation was monitored for behavior and performance in October of 2017. The truck load positioning matched truck loading performed in 2014 so results could be compared. Tables B1 – B7 show the results from 2014 and 2017. In all locations and directions, strains are nearly identical for the data collected in 2017 compared to 2014. This similarity in strains shows that the bridge is behaving in the same manner as it was prior to opening to traffic and implies no cracking of the shear keys.

In addition, the instrumentation was monitored for daily thermal changes from early to late October of 2017. The data showed strain behavior similar to that noted previously.

Table B1: Mid Span Interior Top and Bottom Flange Longitudinal Strains

Load Configuration	Year	Gauge position	Beam 1 (με)	Beam 2 (με)	Beam 3 (με)
One truck on left lane (1)	2014	Top	-47	-32	-38
	2017	Top	-42	-32	-32
	2014	Bottom	45	47	32
	2017	Bottom	44	43	29
One truck on right lane (2)	2014	Top	-26	-16	-27
	2017	Top	-21	-21	-27
	2014	Bottom	25	26	23
	2017	Bottom	26	29	23
Two trucks on mid span (3)	2014	Top	-68	-53	-70
	2017	Top	-62	-53	-64
	2014	Bottom	71	73	55
	2017	Bottom	72	74	52
Two trucks back to back on left lane (4)	2014	Top	-79	-53	-64
	2017	Top	-67	-58	-58
	2014	Bottom	77	73	53
	2017	Bottom	72	74	51

Note: Negative strain constitutes compression

Table B2: Quarter Span Interior Top and Bottom Flange Longitudinal Strains

Load Configuration	Year	Gauge position	Beam 1 (με)	Beam 2 (με)	Beam 3 (με)
One truck on left lane (1)	2014	Top	-31	-16	N/A
	2017	Top	-26	-19	N/A
	2014	Bottom	28	N/A	N/A
	2017	Bottom	27	N/A	N/A
One truck on right lane (2)	2014	Top	-15	-5	N/A
	2017	Top	-12	-15	N/A
	2014	Bottom	18	N/A	N/A
	2017	Bottom	19	N/A	N/A
Two trucks on mid span (3)	2014	Top	-47	-26	N/A
	2017	Top	-40	-32	N/A
	2014	Bottom	47	N/A	N/A
	2017	Bottom	46	N/A	N/A
Two trucks back to back on left lane (4)	2014	Top	-52	-31	N/A
	2017	Top	-44	-34	N/A
	2014	Bottom	49	N/A	N/A
	2017	Bottom	49	N/A	N/A

Note: Negative strain constitutes compression

N/A: gauges were disconnected due to data acquisition's capacity

Table B3: Mid and Quarter Span Longitudinal Strain in Shear Keys 1 and 3

Load Configuration	Year	Shear key 1		Shear key 3	
		Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)
One Truck on Left (1)	2014	-35	-18	-35	-29
	2017	-35	-18	-35	-23
One Truck on Right (2)	2014	-23	-12	-30	-23
	2017	-17	-12	-29	-23
Two Trucks on Mid Span (3)	2014	-58	-29	-71	-52
	2017	-58	-35	-70	-46
Two Trucks on Left (4)	2014	-64	-35	-65	-46
	2017	-63	-35	-64	-46

Table B4: Interior Top Flange Transverse Strains in Beams 1-3

Load configuration	Year	Beam 1 (με)	Beam 2 (με)	Beam 3 (με)
One truck on left lane (1)	2014	22	6	11
	2017	17	6	12
One truck on right lane (2)	2014	6	6	6
	2017	6	6	6
Two trucks on mid span (3)	2014	28	12	11
	2017	21	12	17
Two trucks left (4)	2014	17	12	12
	2017	17	12	17

Table B5: Mid and Quarter Span Transverse Strain in Shear Keys 1-3

Load Configuration	Year	Shear Key 1		Shear Key 2		Shear Key 3	
		Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)
One Truck on Left (1)	2014	6	6	5	3	8	8
	2017	5	2	2	3	5	5
One Truck on Right (2)	2014	2	8	7	7	5	7
	2017	<1	3	3	3	2	4
Two Trucks on Mid Span (3)	2014	8	8	7	6	10	12
	2017	6	5	5	6	7	8
Two Trucks on Left (4)	2014	10	7	6	4	11	13
	2017	9	6	5	6	8	9

Table B6: Mid and Quarter Span Axial Strain in Dowel Bars Embedded in Beams 1-3

Load Configuration	Year	Dowel in Beam 1		Dowel in Beam 2		Dowel in Beam 3	
		Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)
One Truck on Left (1)	2014	17	12	16	N/A	10	N/A
	2017	14	6	6	N/A	8	N/A
One Truck on Right (2)	2014	6	8	12	N/A	4	N/A
	2017	3	3	3	N/A	2	N/A
Two Trucks on Mid Span (3)	2014	17	11	21	N/A	11	N/A
	2017	16	9	9	N/A	9	N/A
Two Trucks on Left (4)	2014	23	18	20	N/A	13	N/A
	2017	21	13	10	N/A	11	N/A

N/A = disconnected gauges

Table B7: Mid and Quarter Span Axial Strain in Dowel Bars Embedded in Shear Keys 1-3

Load Configuration	Year	Dowel in Shear Key 1		Dowel in Shear Key 2		Dowel in Shear Key 3	
		Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)
One Truck on Left (1)	2014	7	4	6	3	10	9
	2017	5	3	4	<1	6	4
One Truck on Right (2)	2014	3	3	4	6	6	6
	2017	<1	1	2	<1	3	<1
Two Trucks on Mid Span (3)	2014	7	6	8	4	13	10
	2017	5	5	6	1	10	5
Two Trucks on Left (4)	2014	9	7	8	3	15	12
	2017	8	5	6	1	10	6

Cost Analysis

Table B8 provides the engineer's estimate and the bids from the contractors for special items for the Sollar's Road Bridge. The special items include the modifications to the box beam shear keys and the grouting of the dowel bar longitudinal joint with UHPC. The special items are a direct additional cost for the new design and hence this data was pulled from the bid tabs. The estimate and bids for the box beams are also included in Table B8. This was done since there was concern that some of the bids might increase beam costs while reducing the special item costs since the grouting is included on the beam estimates for standard designs. As can be seen in Table B8, the bids for the beam modifications were less than the estimated cost for 4 out the 5 bids. It does not appear the difference in the beam modifications was placed into the beam costs for any of the contractors' bids as 3 out of the 5 bids were very close to the engineer's estimate. In fact, the only bid higher on the beam modifications was also higher on the beams cost (Contractor B). For the grouting of the doweled shear keys with UHPC, 3 out of the 5 bids were higher than the estimate. This might be explained as concern with being unfamiliar with the UHPC material and using it in the field.

Table B9 provides the total cost estimate and the bids for the project. The base estimate is the total estimated cost of the project less the cost of the special items. These special items are the delta cost for the project given the new shear key design and the UHPC grouting. The engineer's estimate for this new design was approximately $50,000. The actual estimated cost without this new design was $366,284. The delta cost was estimated to be approximately 14% above the base estimate. However, 3 out of the 5 bids were only approximately 6% above the base estimate and only one bid exceeded the 14% estimate. The percentage of the delta cost relative of the total project cost would likely vary on total project cost and size. Larger total cost projects would likely have a smaller percentage as the costs for changes to the formwork for the larger shear key would be reduced as more beams are cast. Material costs for the UHPC would increase due to larger quantities. However, since this is the first adjacent prestressed concrete box beam bridge in the U.S. utilizing this design, the delta cost is expected to reduce over time if the design is adopted more frequently.

Table B8: Estimated Costs and Bids for Project

	Beam Modifications, per plan		Prestressed Concrete Box Beams, B21-48		Grouting Shear Keys, per plan	
	Unit Cost	Total (7 each)	Unit Cost	Total (7 each)	Unit Cost	Total (372 ft)
Engineer	$2,000	$14,000	$15,000	$105,000	$96.75	$35,991
Contractor A	$1,000	$7,000	$12,100	$84,700	$90.00	$33,480
Contractor B	$2,800	$19,600	$17,500	$122,500	$44.00	$16,368
Contractor C	$665	$4,655	$15,200	$106,400	$114.00	$42,408
Contractor D	$500	$3,500	$14,900	$104,300	$100.00	$37,200
Contractor E	$500	$3,500	$15,000	$105,000	$100.00	$37,200

Table B9: Total Costs and Bids for Project

	Total Project	Base Estimate	% Increase
Engineer	$416,275	$366,284	13.7
Contractor A	$386,777		5.6
Contractor B	$408,146		11.4
Contractor C	$452,761		23.6
Contractor D	$388,403		6.0
Contractor E	$388,057		5.9

Appendix C: LIC 310 Bridge Closure Pour

As previously discussed, the project included multiple phases and widening of the existing bridge. The 9¼" UHPC closure pour existed between the modification of the existing superstructure and an additional shared use pedestrian path. This was done to alleviate concerns related to differential dead load deflection between the modification of the existing superstructure and the additional shared use pedestrian path. The joint had a single No. 5 longitudinal rebar in the top and bottom. Two additional No. 4 longitudinal bars existed in the top of the joint over piers. Transverse No. 5 bars were spaced at 5¾" in the top and bottom and were continuous through the joint.

Traditional joints using UHPC have involved connections between precast elements. In these cases, continuous reinforcement is not possible. Obviously development of bars is not an issue if the bars are continuous through the joint. Since having the reinforcement continuous was possible in the case for this closure pour, it makes sense to have the bars be continuous. In terms of still using the UHPC, it has superior bond capabilities for bond to adjoining concrete to reduce or eliminate bond failure at the interfaces. In addition, UHPC has superior mechanical properties to reduce the possibility or severity of cracking within the joint.

UHPC Pour

The UHPC was mixed and placed in the joint the evening into the night of August 3, 2017. This was done in order to work with the UHPC at lower ambient temperatures to assure set time did not occur too rapidly. The contractor had ice available in case temperatures of the UHPC mix exceeded manufacturer recommendations. Crews added the components into the mixers rented from the supplier. The mixers are high shear mixers to assure proper mixing of the UHPC components. The mix components include a dry bagged Ductal premix material (see right side Figure C1), fibers (see Figure C2), a water reducing admixture, and water. Once properly mixed, the UHPC is flowable as shown in Figure C3. The closure pour joint had an exposed aggregate finish to improve bond between the previously cast decks and the UHPC (see Figure C4). This finish was created by the contractor using a retarder on the forms for the deck and power washing the joint after form removal. Prior to placement of the UHPC the joint was sprayed with a water mist to assure a moist joint surface (see Figure C5). The UHPC was transported from the mixers to the joint by wheelbarrows and placed into a wood chimney. This allowed the UHPC to flow into the joint and form a hydraulic head to fill the joint. The UHPC was placed approximately ¼ in. higher than the surrounding deck surface by anchoring wood strips on each side of the joint. The joint was also sealed on top with plywood to allow the UHPC to flow above the joint and flush with the wood strips on each side of the joint. The UHPC is placed high due to the initial shrinkage of UHPC during setting and to assure no low spots exist from the possibility of trapped air after grinding flush since the UHPC was part of the riding surface. The grinding of the joint occurred approximately 6 weeks after placement of the UHPC. This resulted in the UHPC to have high strength and caused grinding subcontractor to take double the time to grind the length of the bridge for the strip containing the UHPC closure pour.

Figure C1: Preparing to Mix UHPC

Figure C2: UHPC Fibers

Figure C3: UHPC Flow Test

Figure C4: Exposed Aggregate Finish

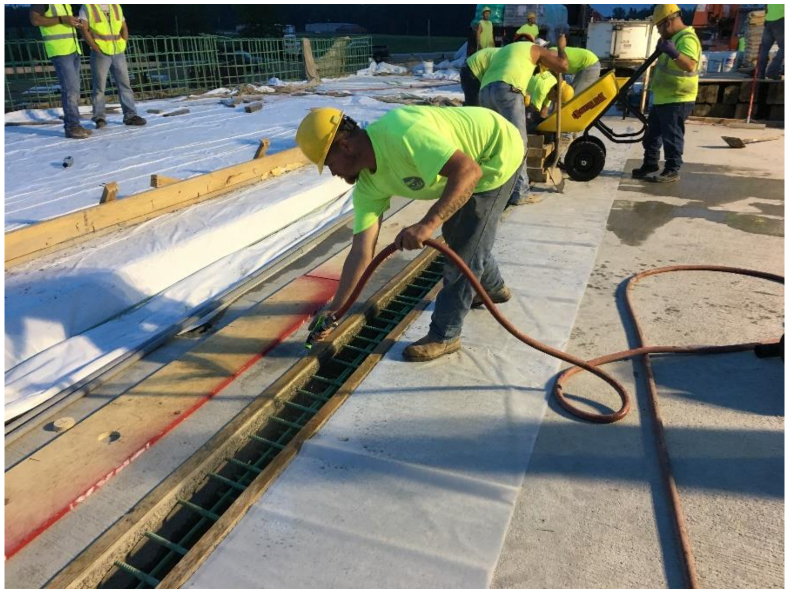

Figure C5: Applying Moisture to Joint

Instrumentation and Data

A total of 15 strain gages were installed at critical locations in the UHPC joint as well as the surrounding deck to monitor performance. These locations can be seen in Figure C6 where Location 1 (L1) was over Pier 1 and Location 2 (L2) was in the center of Span 2. Wires from the installed strain gages were run to the abutment through the deck (up to approximately 100 feet) where data collection was safely accessed.

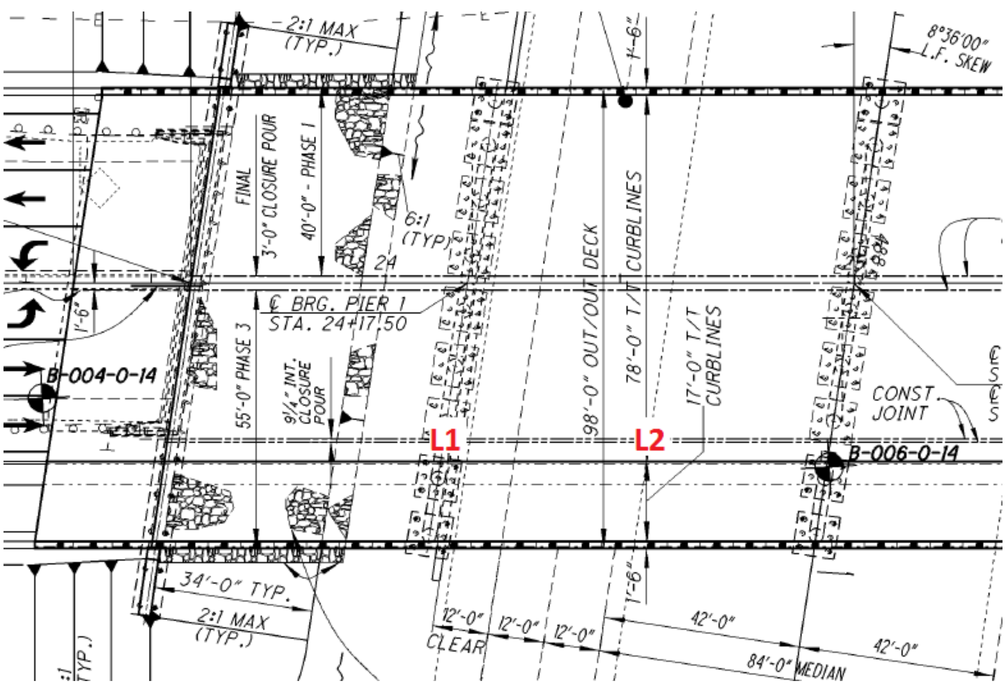

Figure C6: LIC-310-0096 UHPC Strain Gage Installation Locations

At L1, three strain gages were installed on June 21, 2017 in the driving portion of the deck before concrete placement. One gage was installed near the top in the transverse and longitudinal directions and one gage was installed near the bottom in the transverse direction as shown in Figure C7. On July 28, 2017 instrumentation was installed in the UHPC joint. At L1 and L2 locations, strain gages were installed to the top and bottom transverse bars in the UHPC joint as shown in Figure C8. Also strain gages were installed to exist within the UHPC in the top and bottom transverse and longitudinal directions as shown in Figure C9. In addition, thermal couples were installed at numerous locations to measure temperature within the deck and joint as well as ambient temperature.

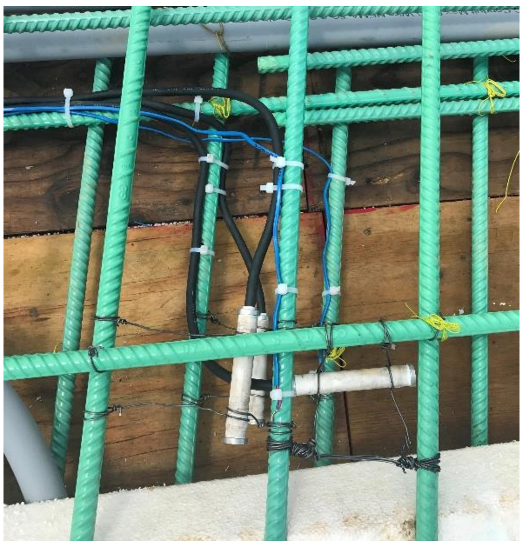

Figure C7: Deck Gages

Figure C8: Rebar Gages

Figure C9: UHPC Gages

Data was collected during UHPC placement the evening of August 3, 2017 and various periods throughout the construction and after opening of the bridge to traffic. Figure C10 shows the strains in the transverse direction at the pier location (L2) in the UHPC and conventional concrete during the UHPC placement. The top of the UHPC obtained large compressive strains approaching 400 microstrain a couple of days after placement. The bottom of the UHPC also showed compressive strains but of lower magnitude. The strains in the adjacent conventional concrete deck changed back and forth from low tensile strains to compressive strains in the top and bottom. Figure C11 provides the longitudinal strains at the pier during and after the UHPC placement. The UHPC in the top and bottom experience tensile strains of similar magnitude but did not exceed 150 microstrain. The conventional concrete in the deck initially showed compressive strains and eventually changed to a variation of low tensile and compressive strains.

Figure C12 shows the strains in the transverse direction at the pier location (L2) in the UHPC and conventional concrete during early March of 2018. The tensile strains were approximately equal and did not exceed 100 microstrain. Compressive strains exceeded 200 microstrain and the top of the UHPC and deck typically showed higher compressive strains than the bottom of the UHPC and deck. Figure C13 provides temperatures during the same time period.

Temperatures in the top of the UHPC and deck were typically more extreme than the bottom of the UHPC and the deck. The higher high and lower low temperatures in the top also occurred slightly before the bottom. The top temperatures occasionally exceeded the ambient temperature measured near the surface of the deck. Figure C14 provides the strain and the temperature at the top of the UHPC in the transverse direction at the pier during March of 2018. As shown in the figure, the strain follows the temperature behavior. Tensile strains are created when temperatures are high and compressive strains occur during lower temperatures. Data collected at other times were similar in behavior.

The highest measured tensile strains in the UHPC occurred longitudinally at the pier not long after placement but did not exceed 150 microstrain. This largest tensile strain level would not be expected to cause cracking in the UHPC.

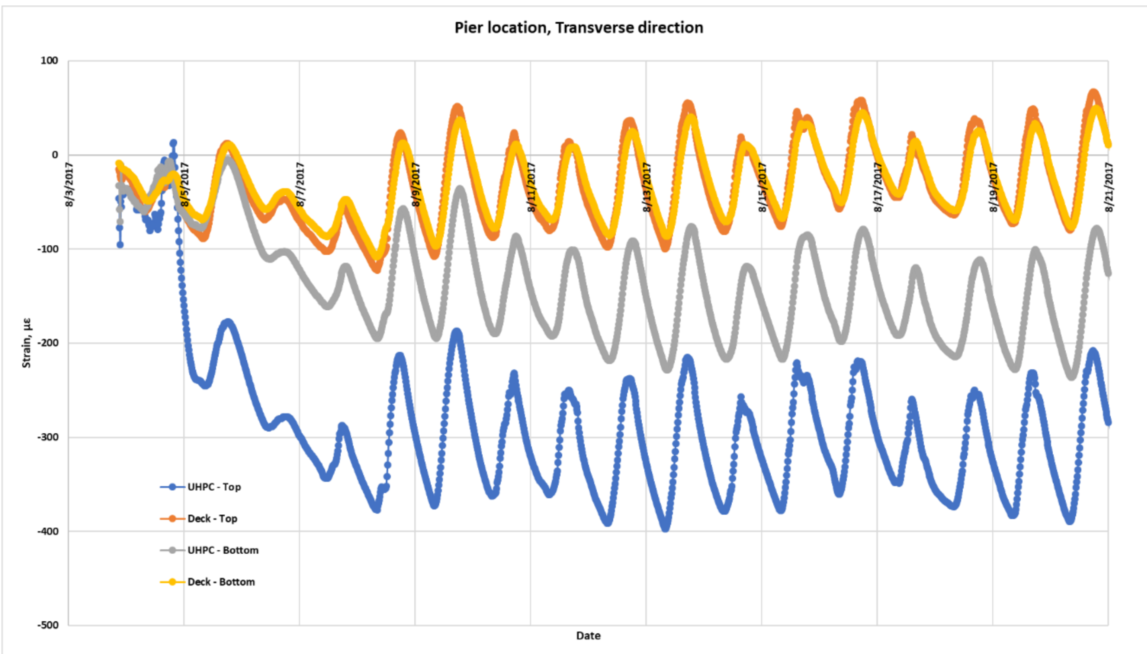

Figure C10: UHPC and Deck Transverse Strains at Pier (August 2017)

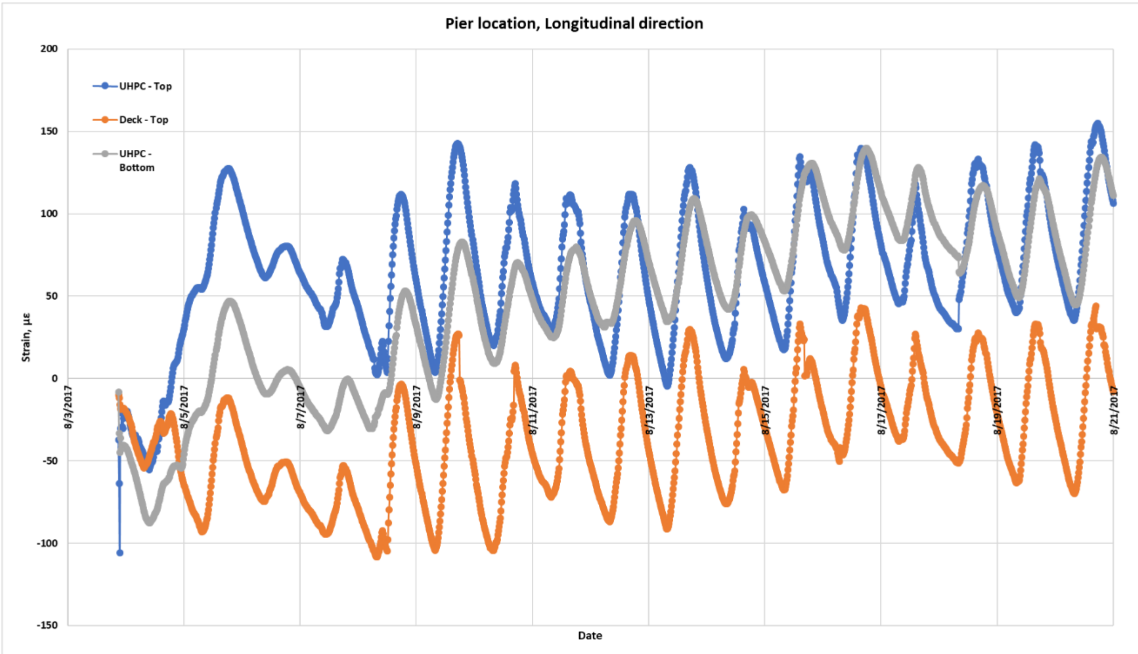

Figure C11: UHPC and Deck Longitudinal Strains at Pier (August 2017)

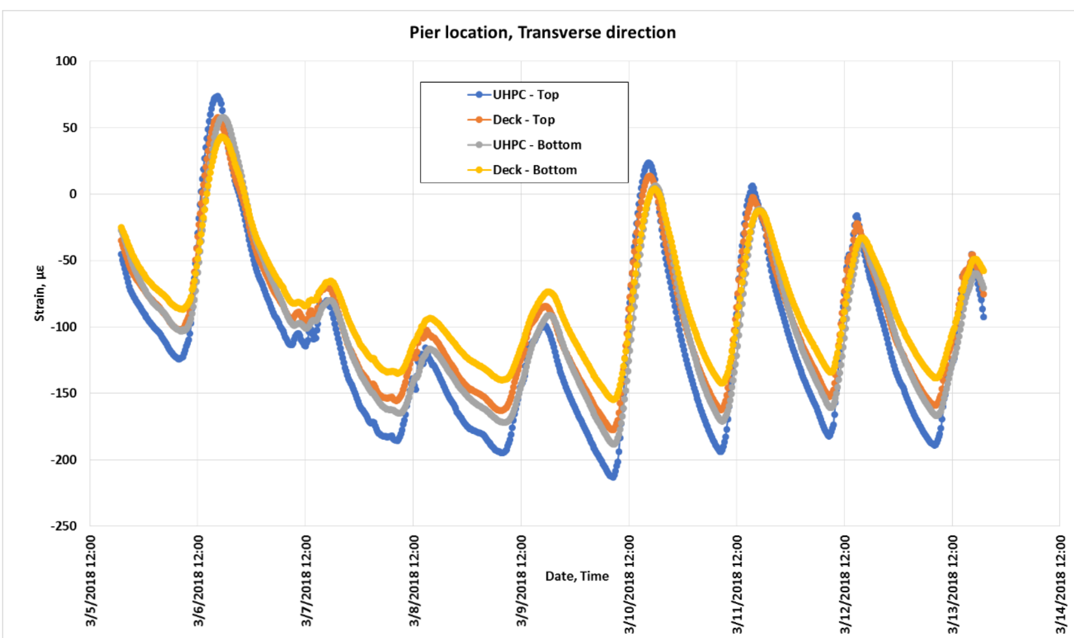

Figure C12: UHPC and Deck Strains at Pier (March 2018)

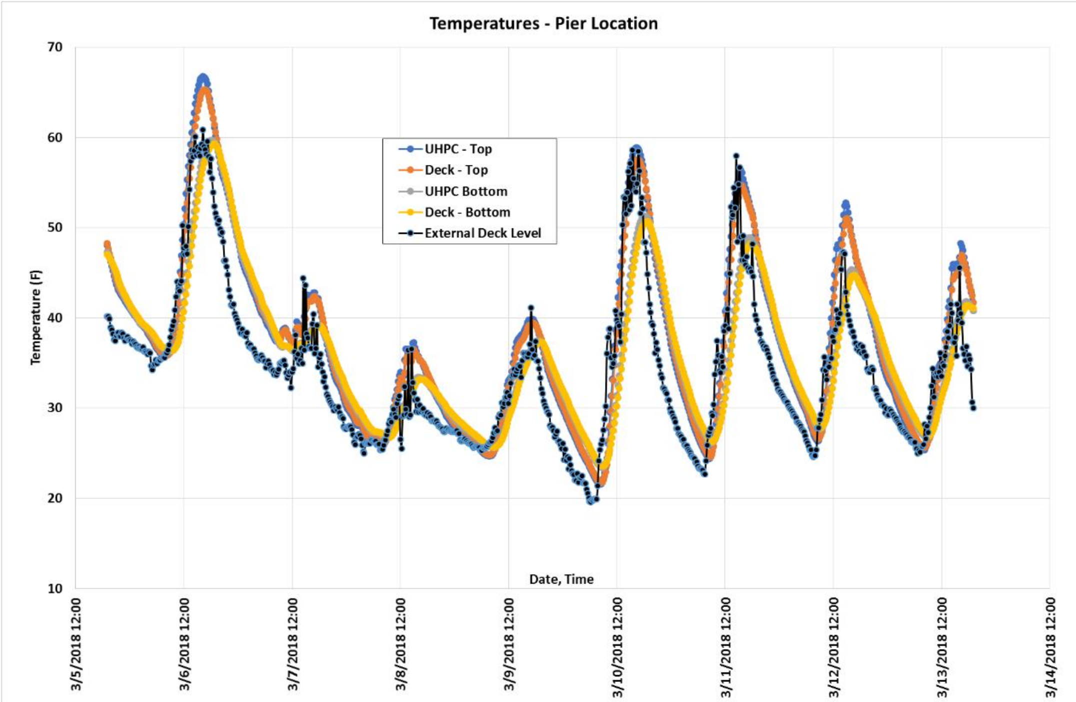

Figure C13: UHPC and Deck Temperatures at Pier (March 2018)

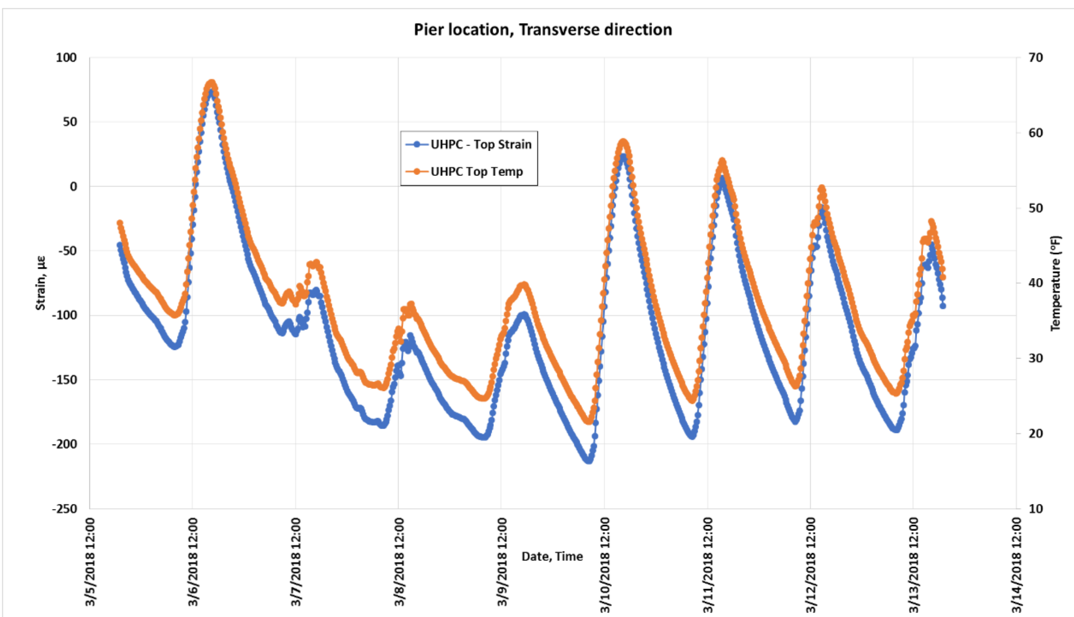

Figure C14: Top UHPC Strain and Temperature at Pier (March 2018)

UHPC Cost

Table C1 provides the bid tab information from the project related to the UHPC. As shown in the table, there was a large variation in the estimated costs related to the PC closure pour. After the project, the contractor that was awarded the project estimated the actual incurred costs after completion of the project to be approximately $4,500/CY. The manufacturer was consulted and believed the cost to be reasonable but noted that the project was small with only 6.7 cubic yards being used and that material shipment, equipment rental and on-site technical services would inflate the overall unit costs. In addition, the UHPC costs relative to the overall project estimates are very small.

Table C1: Contractor Estimated UHPC Costs

Contractor	Estimated Unit Price	UHPC Total	Total Project Estimate
A (Awarded)	$8,000/CY	$53,600	$12,960,321.71
B	$2,100/CY	$14,070	$14,114,125.20
C	$5,000/CY	$33,500	$14,207,383.66

Cracking

Since the cracking was not observed until after the first winter, it is suspected the cracking was the result of restraint from thermal contraction. The cracking was not a concern due to the size of the cracks and because it was sealed by ODOT after observation. Cracks in the deck adjacent to the UHPC appear to be wider as shown in Figure C15. The larger deck crack is above and in between the two smaller circled UHPC cracks in Figure C15. This would tend to imply that fibers were still properly mixed and well distributed within the UHPC. The cracking is not likely due to initial shrinkage/settlement since it was not noticed until spring after the placement of the UHPC and that cracks are also observed in the deck concrete adjacent to the UHPC. The depth of the cracks have likely not penetrated to any significant depth and do not compromise the durability of the UHPC.

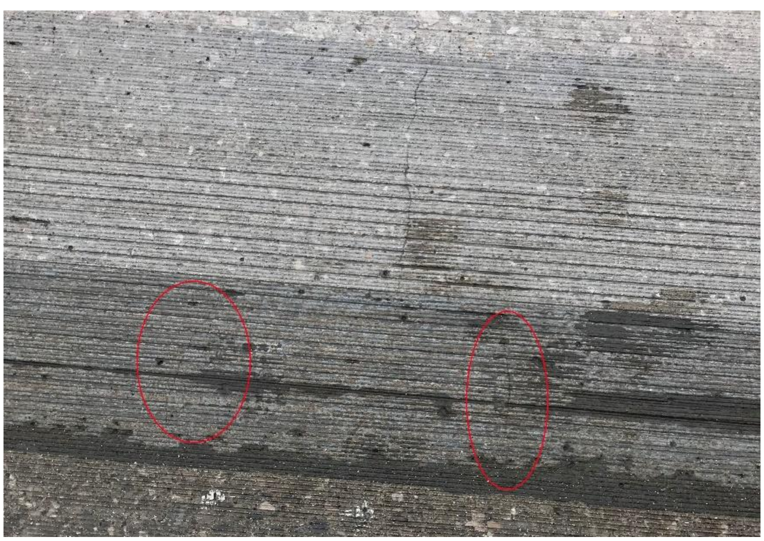

Figure C15: Cracks in UHPC and Deck

Appendix D: GAL 160 Precast Deck Panel System Joints

Panel Details

ODOT District 10 originally planned to use UHPC for the joints between precast concrete deck panels and between the panels and the steel girders on a bridge in Gallia County on Route 160 (GAL 160-18.84). However, the Department ultimately directed to construct a traditional cast-in-place deck for reasons unrelated to UHPC. Details of the original design are still provided here for informational purposes. The three span continuous steel beam bridge had outer spans of 58 ft. and an interior span of 72.5 ft. The bridge was 28 ft. wide. The deck panels were to be full width except near the bridge ends due to the high 35° skew as shown in Figure D1. A total of 21 deck panels were planned to be used for the bridge from 11 different panel details. The full width panels were 8½ in. thick at locations above the girders and 10½ in. thick on the 4 ft. overhangs from the outside girders. Figure D2 shows a plan view of one of the interior panels. Two leveling bolts were provided along each girder line for each panel to adjust elevations and level the panels. Two block outs along each girder line in each panel were to be positioned at girder shear stud locations and filled with UHPC. The transverse cross section of an interior panel is shown in Figure D3. This figure shows the typical 6 ft. 10 in. panel length along the bridge's length.

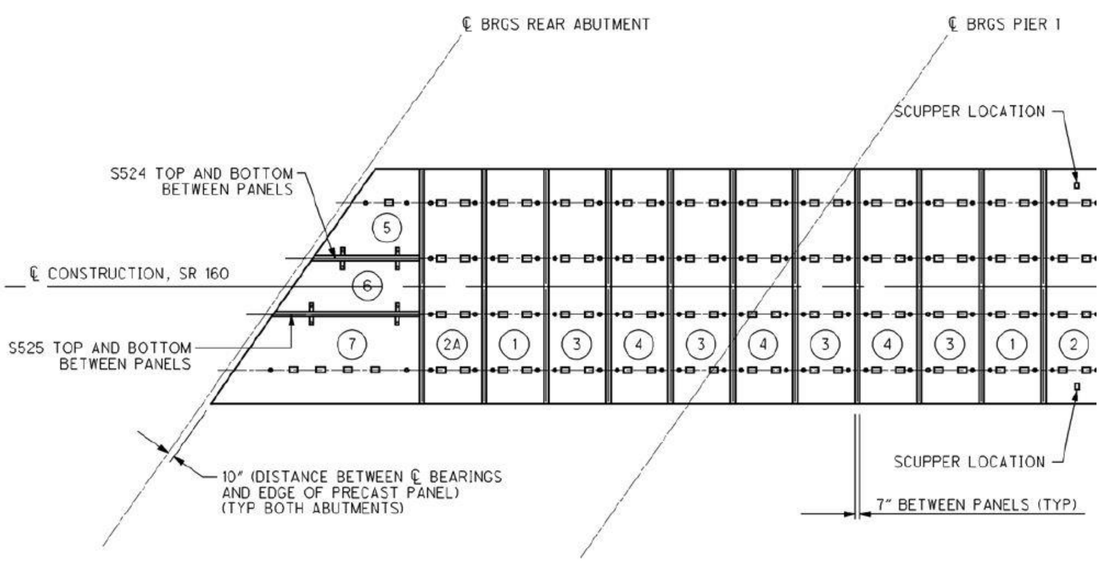

Figure D1: GAL 160 Deck Panels near Rear Abutment

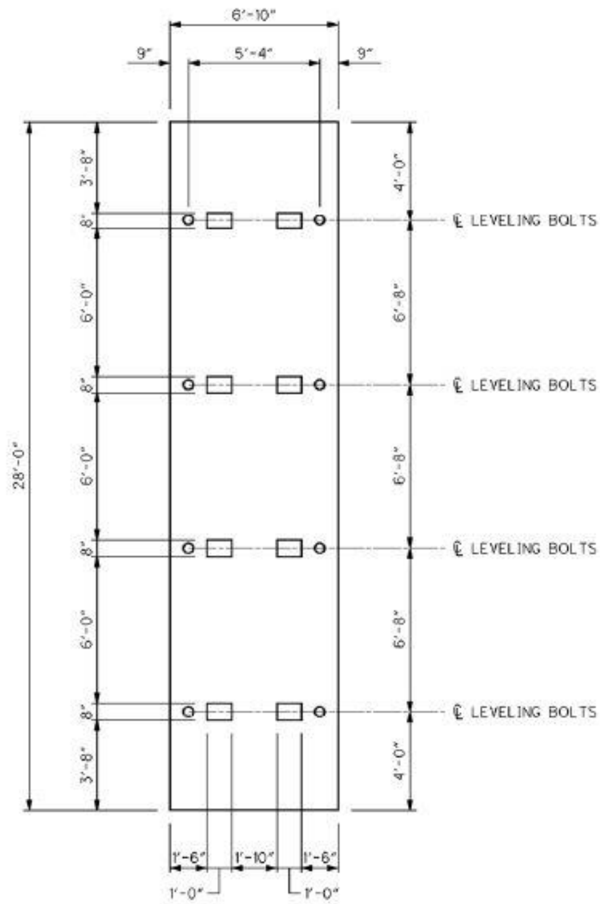

Figure D2: Plan View of Interior GAL 160 Deck Panel

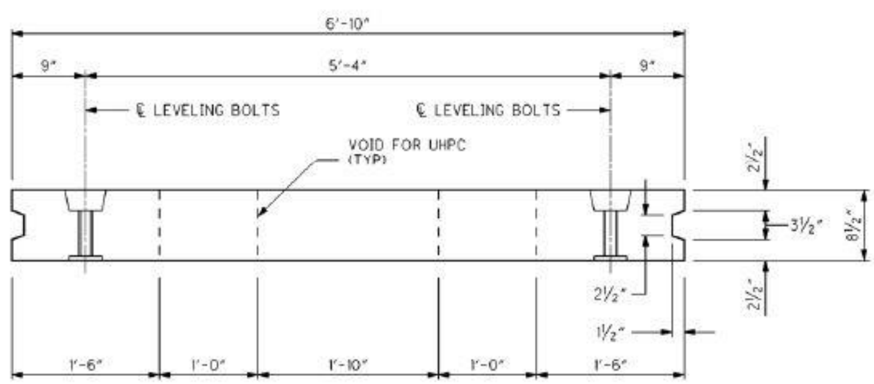

Figure D3: Transverse Cross Section of Interior GAL 160 Deck Panel

Figures D4 and D5 show the reinforcement details for Panel 2. The reinforcement extended 6½ in. into the UHPC joint which are 7 in. wide. The UHPC joint details meet design requirements noted in Technote FHWA-HRT-14-084 (Graybeal, 2014b).

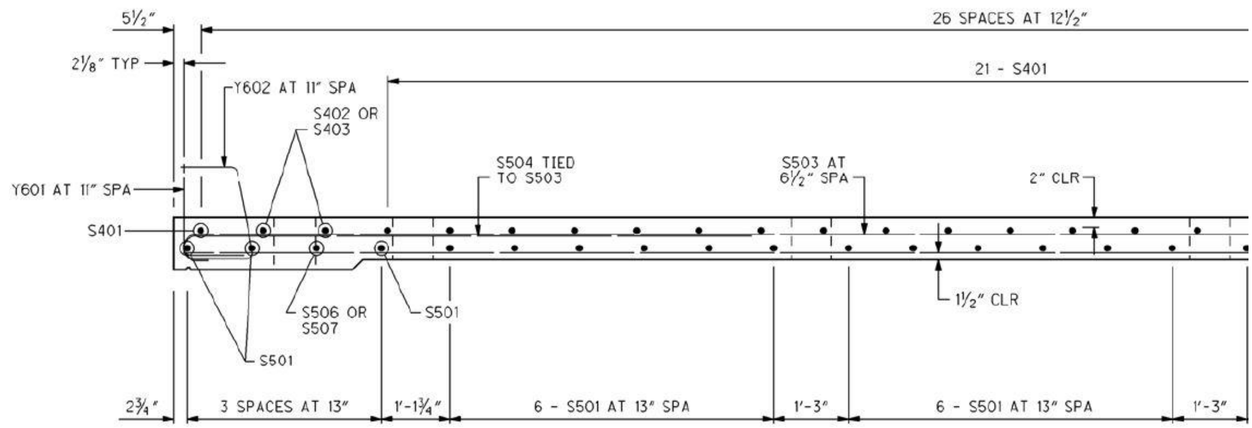

Figure D4: Reinforcement Transverse Cross Section of Interior GAL 160 Deck Panel 2

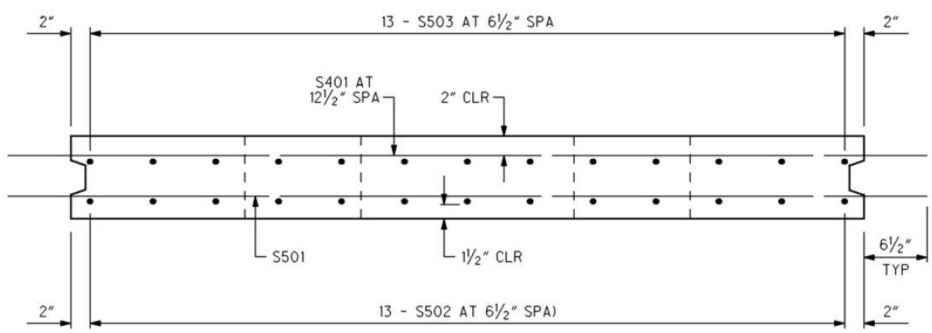

Figure D5: Reinforcement Longitudinal Cross Section of Interior GAL 160 Deck Panel 2

The end panels over the girders had a unique connection design due to the high skew. Figure D6 shows end panel 5, and Figure D7 shows the detail of the longitudinal connections of the end panels over the girders.

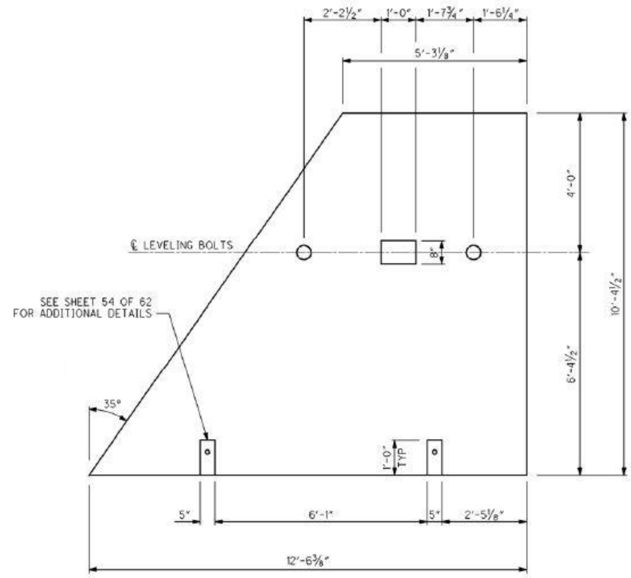

Figure D6: Plan View of GAL 160 End Deck Panel 5

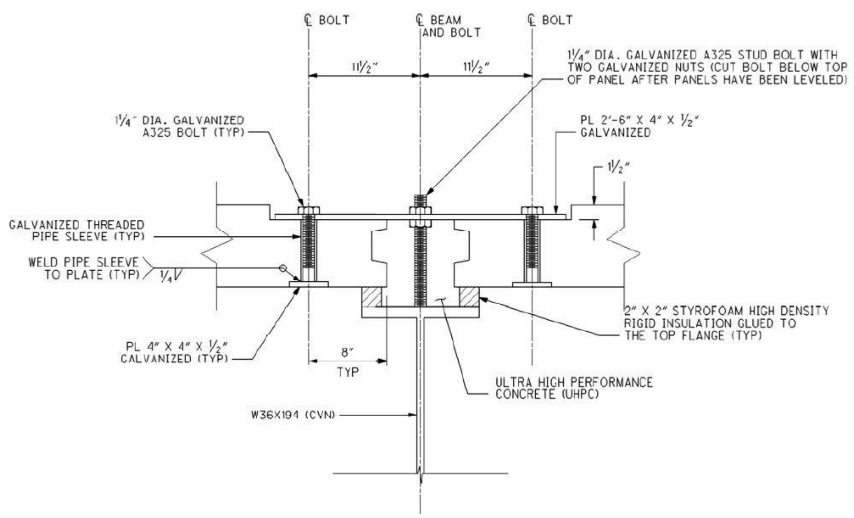

Figure D7: GAL 160 End Panel Connection Detail over Girder