Design and Construction of UHPC-Based Bridge Preservation and Repair Solutions

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FOREWORD

In 2021, ultra-high performance concrete (UHPC) for bridge preservation and repair (P&R) was rolled out as one of the innovative technologies in the Federal Highway Administration's Every Day Counts program. UHPC has been a proven solution in bridge construction for connections between prefabricated bridge elements and is an emerging solution for P&R that offers enhanced performance and improved lifecycle cost over traditional methods. UHPC repair solutions are long lasting and resilient, requiring less maintenance and fewer follow-up repairs than conventional methods. The information presented in this document provides background, context, and foundational knowledge to bridge owners and designers interested in using this innovative solution for preserving our Nation's highway bridges.

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16. Abstract

This report is intended for bridge owners, contractors, and their supporting professionals responsible for design, construction, materials, and maintenance who are interested in including ultra-high performance concrete (UHPC) in their bridge preservation and repair (P&R) toolkit. This document aims to familiarize the reader with the material mechanical and durability properties of UHPC, along with common and emerging UHPC-based P&R solutions. Most notably, the document contains design and construction recommendations for three promising and fastest growing UHPC P&R applications: bridge deck overlays for rehabilitation, link slabs, and steel beam end repair. These recommendations are limited in scope but provide valuable information for all owner agencies considering the development of materials, construction, and design specifications. Lastly, much of the information provided herein builds on previous UHPC design and construction documents published by the Federal Highway Administration in *Design and Construction of Field-Cast UHPC Connections* (Report No. FHWA-HRT-19-011).

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1,000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2,000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2,000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	2.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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CHAPTER 1. INTRODUCTION

Ultra-high performance concrete (UHPC) is emerging as a promising solution for bridge preservation and repair (P&R), as it offers an effective and durable solution for extending the service life of existing infrastructure, minimizing impact on the end users, and maximizing the value of agency investments. Field-deployed UHPC P&R solutions can arrest further infrastructure deterioration by enhancing the durability. These solutions can also facilitate strengthening options for aging infrastructure as well.

OBJECTIVE AND USE

This report is intended for bridge owners, contractors, and their supporting professionals responsible for design, construction, materials, and maintenance who are interested in including UHPC in their bridge P&R toolkit. This document aims to familiarize the reader with key concepts related to UHPC, such as its material mechanical and durability properties, along with common UHPC-based P&R solutions. It also gives an overview of some emerging UHPC-based solutions for bridge P&R. Most notably, the document contains design and construction recommendations for three promising and fastest growing UHPC P&R applications: bridge deck overlays for rehabilitation, link slabs, and steel beam end repair. These recommendations are limited in scope but provide valuable information for all owner agencies considering the development of materials, construction, and design specifications. Lastly, much of the information provided herein builds on previous UHPC design and construction documents published by the Federal Highway Administration (FHWA) (Graybeal 2019; Graybeal and Leonard 2018).

BRIDGE PRESERVATION AND REPAIR

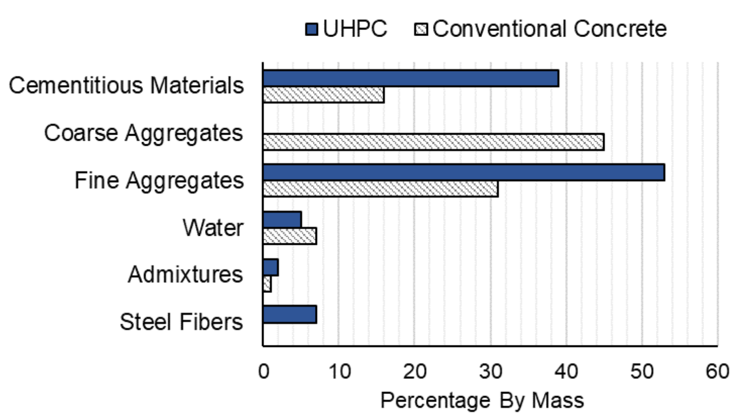
Defining the terms preservation, repair, and the like is important in the context of this document. Bridge preservation is a proactive approach to extending bridge service life and is defined as "…actions or strategies that prevent, delay, or reduce deterioration of bridges or bridge elements; restore the function of existing bridges; keep bridges in good or fair condition; and extend their service life. Preservation actions may be cyclic or condition-driven" (FHWA 2018). This approach can slow the progress of deterioration and extend its life. Preservation is a long-term strategy adopted by an agency to reduce the lifecycle cost of a structure and is based on the principle that the repair cost is proportional to the deterioration level. Repair refers to the treatment provided to a structure, component, or structural element that has lost its functionality. Repair is a broad term that comprises maintenance treatments to restore a minimum structural capacity, strengthening to add structural capacity, or rehabilitation to meet specification requirements. Maintenance may involve preventive-, cyclical-, or condition-based treatments. Rehabilitation involves major work required to restore the bridge structural integrity and correct major safety defects (FHWA 2018).

CHAPTER 2. UHPC-CLASS MATERIALS

UHPC is a fiber-reinforced, portland cement-based product with advantageous fresh and hardened properties. Through advancements in superplasticizers, dry-constituent gradation, fiber reinforcements, and supplemental cementitious materials, UHPC outperforms conventional concrete. Developed in the late 20th century, this concrete class has emerged as a capable replacement for conventional structural materials in a variety of applications, including those related to bridge P&R. An alternative name for UHPC is ultra-high performance fiber-reinforced concrete (UHPFRC). For this document, UHPFRC is synonymous with UHPC since fiber reinforcement is a key component of UHPC-class material. The following subsections provide summaries of constitutive materials, commercial availability, material properties, and existing transportation infrastructure deployments.

CONSTITUTIVE MATERIALS

Common UHPC formulations consist of several solid powders, including portland cement, fine silica sand, finely ground quartz flour, and microsilica (silica fume). Additional constituents include high-range, water-reducing admixtures, water, and microfiber reinforcement. For structural applications, fibers are commonly composed of drawn and cut steel wires that measure 0.5–0.8 inches in length and 0.03–0.05 inches in diameter and have tensile strength between 100 and 300 ksi. UHPCs formulated for nonstructural application may use polymeric or synthetic fibers instead of steel fiber. The quantities of these constitutive materials are engineered to form an optimized gradation of granular constituents, a water-to-cementitious materials ratio less than 0.28, and a steel fiber dosage of at least 2.0 percent by volume to promote strain-hardening behavior in tension. In addition to the basic material constituents, the mix design may be modified with additives or admixtures to produce a desired performance characteristic, such as increased flowability, thixotropy, longer set time, reduced heat of hydration, or higher early strength. As a point of reference, figure 1 shows a comparison between the quantities of constituents, by mass, for conventional concrete and UHPC. The reader is referred to *Design and Construction of Field-Cast UHPC Connections* for additional information on UHPC constituents (Graybeal 2019).



Source: FHWA.

Figure 1. Graph. Comparison of the constituents, by mass, between conventional concrete and UHPC.

MATERIAL PROPERTIES

UHPC is a class of materials. As such, different mix designs will lead to different performance attributes. Table 1 presents the observed range of performance for a suite of properties, including fresh, mechanical, durability, and dimensional stability. The data presented in this table were generated by research conducted at FHWA's Turner-Fairbank Highway Research Center (TFHRC). In terms of the design and construction guidance provided in this document, some properties hold greater relevance than others, depending on the specific project requirements and the performance objectives of the P&R action. Properties beyond those presented in table 1 can be achieved.

Table 1. Expected range of material properties of field-cast UHPC.

Property	Test Method and Details	Expected Range
Unit weight	ASTM C642 (ASTM 2013a)	145–160 lb/ft^3
7-day compressive strength	ASTM C1856 (ASTM 2017a) ASTM C39 (ASTM 2020a)	14–20 ksi
14-day compressive strength	ASTM C1856 (ASTM 2017a) ASTM C39 (ASTM 2020a)	18–22 ksi
Modulus of elasticity	ASTM C1856 (ASTM 2017a) ASTM C469 (ASTM 2014a)	5,600–8,000 ksi
Poisson's ratio	ASTM C1856 (ASTM 2017a) ASTM C469 (ASTM 2014a)	0.1–0.2
Direct tension cracking strength	FHWA-developed direct tension test (Graybeal and Baby 2013)	0.75–1.2 ksi*
Direct tension postcracking strength	FHWA-developed direct tension test (Graybeal and Baby 2013)	0.75–1.2 ksi*
Direct tension strain capacity	FHWA-developed direct tension test (Graybeal and Baby 2013)	0.0025–0.006 (inches/inches)
Direct tension bond strength	ASTM C1583, bonded to an exposed aggregate surface (ASTM 2020b)	0.35–0.6 ksi
Long-term drying shrinkage	ASTM C1856 (ASTM 2017a) ASTM C157 (ASTM 2017b)	0.0003–0.0012 (inches/inches)
Long-term autogenous shrinkage	ASTM C1856 (ASTM 2017a) ASTM C157 (ASTM 2017b)	0.0002–0.0009 (inches/inches)
Chloride ion permeability	ASTM C1856 (ASTM 2017a) ASTM C1202 (ASTM 2019) 56 days after placement	50–500 Coulombs
Freeze-thaw resistance	ASTM C1856 (ASTM 2017a) ASTM C666 (ASTM 2015a) After 600 cycles	Relative dynamic modulus of elasticity > 95 percent

Property	Test Method and Details	Expected Range
Initial set time	ASTM C403 (ASTM 2016)	4–10 hours
Final set time	ASTM C403 (ASTM 2016)	7–24 hours
Alkali-silica reaction	ASTM C1260 or ASTM C1567 (ASTM 2013b, 2014a) Tested for 28 days	Innocuous

*The expected range of values for direct-tension, sustained, postcracking tensile strength is the same as the expected range of values for direct-tension cracking strength. This equivalence is because, for a strain-hardening material like UHPC, the minimum value of direct-tension, sustained, postcracking tensile strength is the direct-tension cracking strength.

AVAILABILITY

UHPC availability in the U.S. market has increased dramatically over the last decade. This increase is directly associated with UHPC's growing popularity as an option for construction of the built environment. For this document, UHPC availability is categorized as follows:

- Commercial material suppliers: Companies and/or organizations that have completed research and development and have a UHPC mixture(s) on the commercial market are included in this category. Products in this category could include, but are not limited to, prebagged or ready-mix-type products. Also included are products for which a license is granted for third-party use. In all, the mixtures should be effectively proprietary. In some cases, commercial material suppliers also provide onsite project assistance. This assistance includes, but is not limited to, qualified technicians who can provide onsite batching, mixing, and testing support to the UHPC installer.

- Fabricated product suppliers: Companies and/or organizations that supply fabricated products composed of UHPC for the built environment are included in this category; for example, a precaster of bridge girders who offers girders composed of UHPC. These companies and/or organizations may have developed their own proprietary UHPC mixtures for use in their products, or they could source their materials from commercial material suppliers.

- Open-source mixtures: UHPC mixtures that have been developed by individuals, research teams, companies, and/or organizations, where the mix design is available in the public domain and is free of charge to use, are included in this category. These mixtures could also be referred to as nonproprietary UHPC mixtures.

Multiple States, typically in cooperation with a local university, have also worked toward developing UHPC mixtures using local raw ingredients to lower the material cost and promote use in the State (Berry, Snidarich, and Wood 2017). In some cases, these locally developed mixtures have been deployed on bridge construction projects (El-Tawil et al. 2018). FHWA also has information available that discusses developing nonproprietary, regional UHPC mixtures (Wille and Boisvert-Cotulio 2013). The direct costs of a nonproprietary mixture may be as much as 50 percent less expensive relative to commercial mixes (Wille and Boisvert-Cotulio 2013). However, this price does not include other necessary costs, such as research and development, quality control (QC), blending, and packaging, which could be significant.

DEPLOYMENT IN U.S. BRIDGE CONSTRUCTION

UHPC has been used in the United States for bridge construction since 2006. Through 2021, more than 350 applications of UHPC have occurred on bridge construction projects in the United States (FHWA 2022). At the time of this publication, the most common bridge construction application of UHPC in the United States was for connections between prefabrication bridge element and systems. UHPC use in connections is attractive due to its superior durability and its ability to make the short reinforcing bar lap splices within the connections, which enhance constructability. Examples of these element and systems include, but are not limited to, precast concrete deck panels, box beams, precast modular bridge units, connections between existing columns and new, precast bent caps, and connections between precast abutment elements (Graybeal 2019). Other UHPC applications in new construction in the United States include precast, prestressed bridge girders, foundation piles, and precast UHPC bridge decks (Blais and Couture 1999; Sritharan 2015; Aaleti, Petersen, and Sritharan 2013). As it relates to the subject of this report, between 2013 and 2020, more than 40 U.S. bridges have employed UHPC for preservation, repair, and/or retrofit applications. This number includes some UHPC link slabs constructed on new bridges.

CHAPTER 3. PROMISING APPLICATIONS OF UHPC FOR PRESERVATION AND REPAIR

UHPC offers an effective, durable, and versatile solution for bridge P&R. To date, several different UHPC applications for bridge P&R have been deployed in the field. The objective of this section is to provide an overview of some of the emerging and promising UHPC P&R solutions. The coverage of each solution aims to include a basic description of the technique, some basic design considerations, and drawings and/or photos of the solution. Three very promising UHPC P&R applications are presented in detail at the end of the section: bridge deck rehabilitation using UHPC overlays, expansion joint replacement with UHPC link slabs, and corroded/deteriorated steel beam end repair using UHPC encasement.

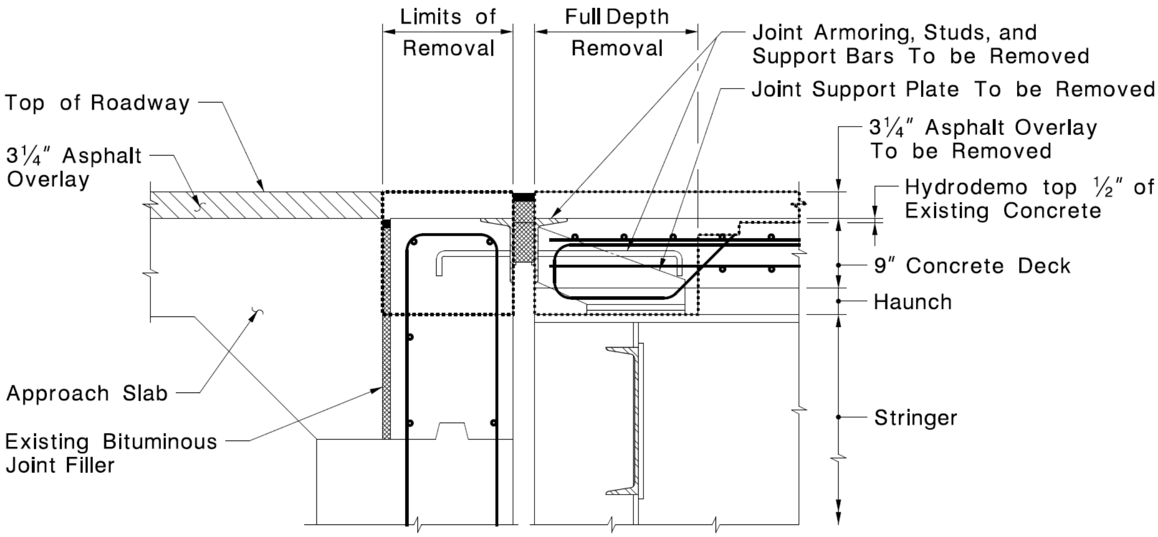
UHPC HEADERS FOR BRIDGE DECK JOINTS

Cracking and deterioration of bridge deck joint headers is a common issue on bridges with high truck traffic volumes and/or relatively flexible superstructures. Conventional headers are typically composed of conventional or elastomeric concretes. They commonly employ steel angles or channel sections to armor the transition between the joint. A typical armored expansion joint header is shown in figure 2. This header has been in service and exhibits some deterioration of both steel and concrete. Joint headers constructed using UHPC are expected to last longer and do not necessarily require using steel sections for joint armoring, which simplifies construction.

Source: FHWA.

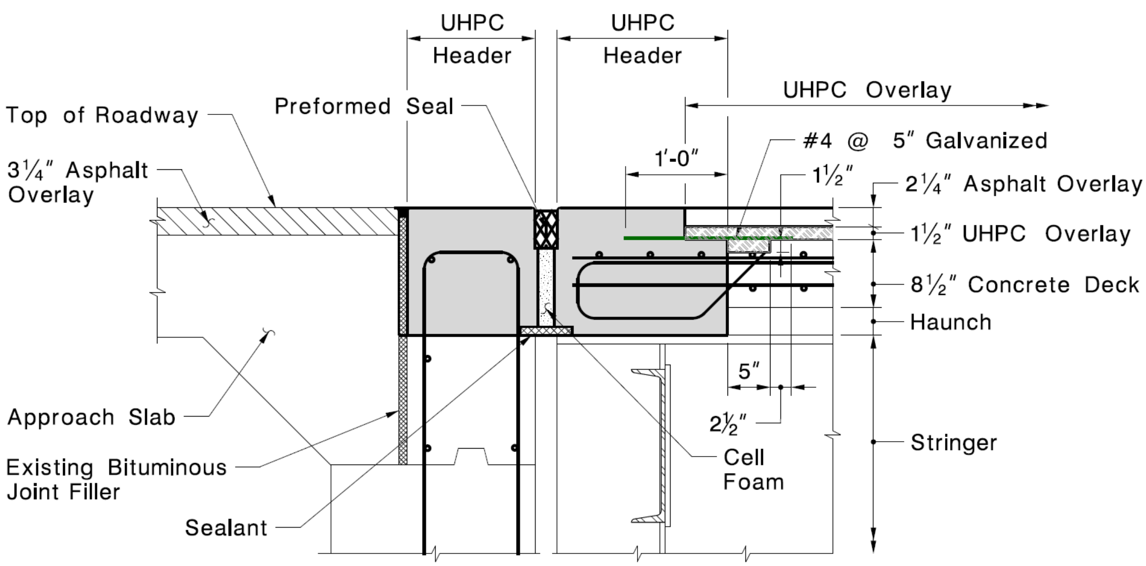
Figure 2. Photo. Traditional armored expansion joint header.

New Jersey Department of Transportation (DOT) has used this solution to rehabilitate the expansion joints in two different bridges: I–295 Northbound Bridge over Mantua Creek in Paulsboro, NJ, and I–280 Westbound (WB) Bridge over the Newark Turnpike in Kearny, NJ. Figure 3 shows the details for the headers on either side of the expansion joint for the I–280 bridge as they existed before replacement. Figure 4 shows the details of the UHPC headers that were installed. The UHPC headers were approximately 15 inches deep and were cast using a self-leveling and self-consolidating UHPC mixture, similar to that commonly used for field-cast UHPC connections. The final configuration at the expansion joint does not include steel header angles typically found on deck joint headers. UHPC's high strength and abrasion resistance are expected to be able to sustain the demand of repeated wheel loads and snowplow impacts.



Source: FHWA.

Figure 3. Illustration. Existing joint header removal detail: I–280 WB over Newark Turnpike.



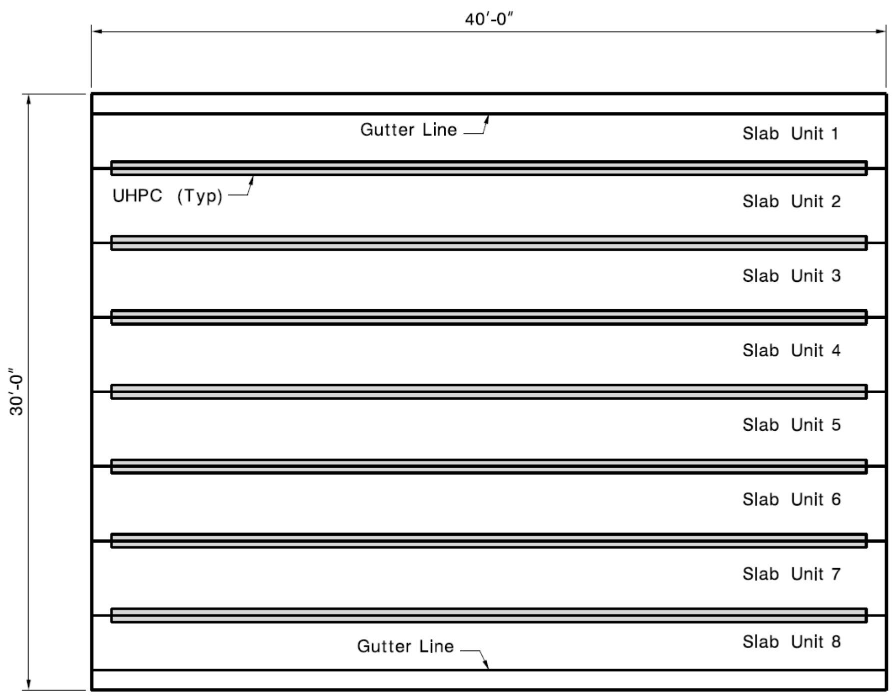
Source: FHWA.

Figure 4. Illustration. UHPC joint header repair detail: I–280 WB over Newark Turnpike.

REPAIR OF PREFABRICATED BRIDGE ELEMENT CONNECTIONS

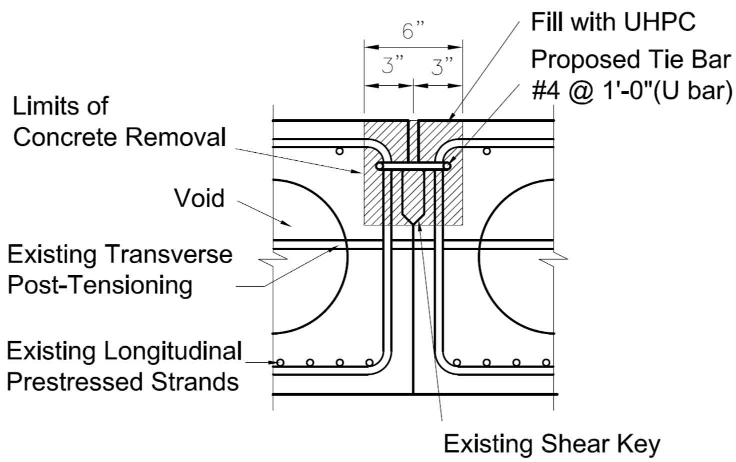
Using prefabricated bridge elements has been common in U.S. highway bridge construction for many decades. These systems commonly use field-cast connections to create continuity between structural elements. UHPC is a known solution for creating robust and durable connections between prefabricated bridge elements in new construction. It can also be a viable solution to repair existing connections that exhibit deterioration or leakage or elements that exhibit differential deflection due to poor connections. For example, adjacent precast concrete box beam and voided slab bridges have a history of durability issues related to longitudinal cracking along shear key connections.

In 2016, Florida DOT (FDOT) rehabilitated the Martin Downs Boulevard Bridges over Danforth Creek in Palm City, FL, by removing the existing traditional partial-depth grouted shear keys and replacing the removed concrete with UHPC. The objective was to create a new, durable connection and restore composite action between the beams, thus eliminating differential deflections between elements. As shown in figure 5, the existing shear keys, along with the surrounding concrete, were removed, exposing the shear reinforcement within the voided slab beams. The reinforcement in the adjacent slabs was connected with horizontal stirrups, and the excavated regions were filled with UHPC. Figure 6 shows the UHPC being installed on this project. A similar project was completed in St. Clair County, MI, on Kilgore Road Bridge over Pine River in Kenockee Township. Here, UHPC was used to repair joints between the bridge's precast double T-beams. UHPC installation on this project is shown in figure 7. Also, this project was the first to deploy an open-source UHPC mixture on a U.S. bridge.



Source: FHWA.

A. Plan view of the planned repair.



Source: FHWA.

B. Details of the repair.

Figure 5. Illustrations. UHPC connection repair used on the Martin Downs Boulevard Bridges.

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Figure 6. Photo. Installation of UHPC on one of the Martin Downs Boulevard Bridges.

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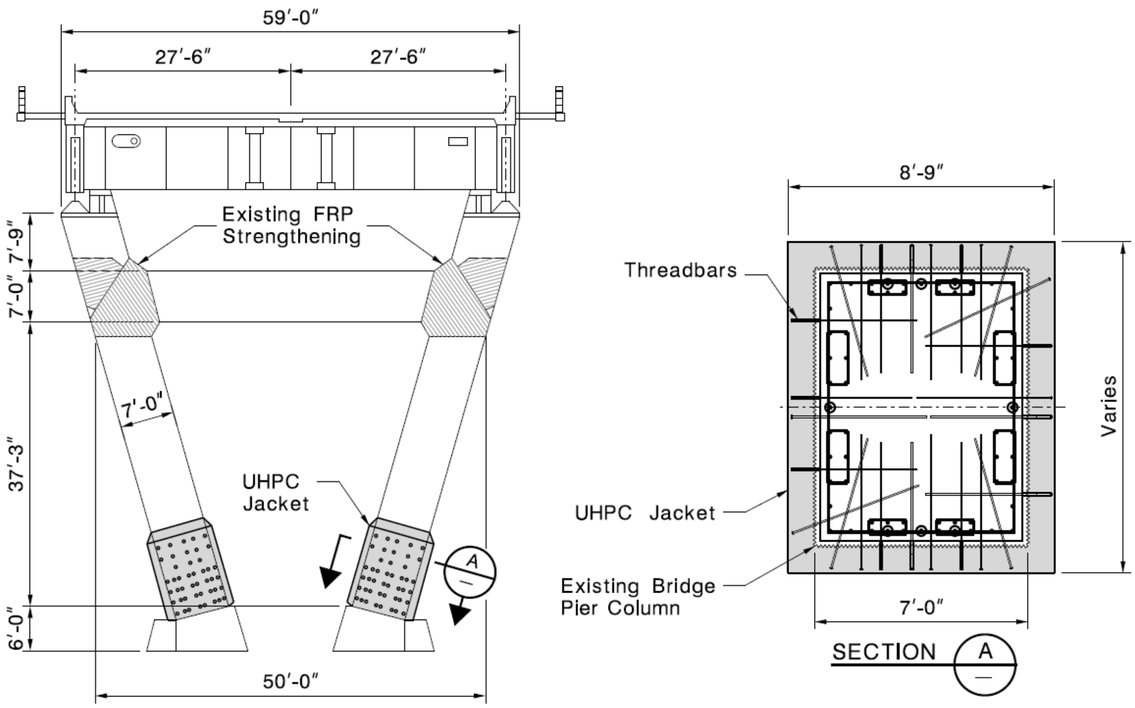
Figure 7. Photo. Installation of UHPC connection repair project on the Kilgore Road Bridge over Pine River in Kenockee Township, MI.

SEISMIC RETROFIT

Bridge structures built before the establishment of modern seismic bridge design and detailing provisions often require upgrading or retrofitting to enhance their seismic performance. Commonly, the reinforced concrete columns of these structures require the most attention, given that the columns are typically the primary lateral load-resisting elements in the structures. Traditionally, structural steel, fiber-reinforced polymer (FRP), or bulky reinforced concrete jackets have been employed to upgrade the strength and ductility of seismically deficient bridge columns. UHPC provides an alternative column-strengthening or -jacketing solution to these traditional methods. Laboratory research has demonstrated that UHPC can restore bridge column capacity with deficient reinforcing bar lap splices located in bridge column plastic hinge zones (Dagenais, Massicotte, and Boucher-Proulx 2018).

In 2014, the British Columbia Ministry of Transportation used UHPC jackets to encase and confine the hinge zones of pier columns on Mission Bridge in Mission, British Columbia, Canada. Built in 1973, the bridge was found to have multiple seismic vulnerabilities. As such, the bridge had previously used FRP wraps to retrofit the plastic hinge zones. One such seismic vulnerability was the threat of lateral spreading in specific pier locations. While ground improvements in the form of deep compaction piles mitigated the issue at most pier locations, a single pier required additional strengthening. For this location, a UHPC jacket was selected because it would provide an aesthetically pleasing and cost-effective retrofit solution compared with other alternatives. The construction procedure included removing the existing FRP wraps, after which the column concrete surfaces were roughened and steel rods were installed to anchor the UHPC to the surface of the columns. Steel stirrups were added around the column perimeters, and then the stirrups and the anchors were encased in a 9-inch-thick jacket of UHPC. The repair schematic is shown in figure 8, and the finished product is shown in figure 9.



Source: FHWA.

Figure 8. Illustration. Schematic of the UHPC seismic retrofit for the Mission Bridge.

Figure 9. Photo. Completed seismic retrofit for the Mission Bridge using UHPC.

COLUMN AND PIER WALL REPAIRS

Cast-in-place UHPC can be used to repair deteriorated bridge columns and pier walls. This application is similar to the seismic retrofit of the Mission Bridge, except that it is not focused on a plastic hinge region or ductility enhancement. The objective is rather to enhance the strength and durability of the original bridge pier. This process would normally require removing poor cover concrete, roughening the concrete substrate to achieved good bond with the UHPC, and replacing the removed concrete with UHPC. This technique was deployed on a reinforced concrete pier wall in 2014 on a Canadian National Railway (CN) Bridge in Quebec, Canada, as shown in figure 10. In the case of the CN Rail Bridge, the concrete cover was removed, exposing steel piles that had been embedded in the pier. All existing reinforcing was exposed and evaluated. Heavily corroded reinforcement was replaced with new reinforcement anchored into the pier's concrete core, while reinforcement that exhibited acceptable levels of corrosion was left in place. Formwork was installed around the entire pier wall perimeter and was constructed such that the reconstructed wall would take the shape of the original pier geometry. UHPC was then poured into the formwork to encapsulate the existing core and all of the reinforcing, thus restoring the pier to a like-new condition, as shown in figure 11.

Figure 10. Photo. CN Rail Bridge with concrete cover removed from pier.

Figure 11. Photo. Completed UHPC pier repair for CN Rail Bridge.

CONCRETE ELEMENT PATCHING

Prestressed concrete beams experience cracking and spalling at their ends due to improper protection from water and deicing chemicals at the deck joints. Such exposure leads to deterioration at the beam ends. Furthermore, situations arise in which concrete elements are damaged during construction or erection and require repair. Laboratory research has demonstrated that UHPC can be a viable solution for repairing and strengthening concrete beams that have undergone damage to the beam ends (Shafei, Phares, and Shi 2020). In 2017, FDOT deployed UHPC to repair a spliced U-girder (Haber and Graybeal 2019). During construction, the concrete that was installed as a closure pour for the midspan girder-to-girder splice was poorly consolidated and, thus, needed repair. The region to be repaired was highly congested; thus, the repair material needed to be highly flowable. UHPC was selected as the repair material because it bonds very well with steel reinforcement and existing concrete, but also because it is highly flowable and self-leveling. The poorly consolidated concrete was removed and replaced with UHPC. Colorado DOT has also deployed UHPC for concrete element patching of bridge decks (FHWA 2021).

SPRAYABLE UHPC

Sprayable UHPC is applied by pumping and spraying UHPC in a similar manner as that for traditional shotcrete. Figure 12 shows a nozzleman applying sprayable UHPC to a concrete panel during the research and development phase of product development. The advantages of UHPC shotcrete are consistent thickness throughout the operation; rapid cure times; high strength, which can eliminate the need for embedded reinforcement; and enhanced durability. The primary advantage of applying UHPC by spraying versus forming and pouring is the elimination of formwork. Scenarios where UHPC shotcrete could be advantageous include, but are not limited to, repair of tunnels, culverts, bridge piers, and dams. At the time of this writing, sprayable UHPC has not yet been deployed in the United States but has been deployed in Europe (Doiron 2019).

Figure 12. Photo. Nozzleman applying sprayable UHPC.

BRIDGE DECK OVERLAYS FOR PRESERVATION, STRENGTHENING, AND REHABILITATION

Background

Highway bridge decks require much attention from bridge owners, and bridge deck deterioration is a common issue in many States. UHPC is a very promising material for bridge deck preservation, strengthening, and rehabilitation when used as a bridge deck overlay. In the context of this document, a bridge deck overlay is defined as a layer of material that is placed atop a bridge deck, with or without removing any of the existing bridge deck concrete. The performance objectives of a bridge deck overlay vary. In some cases, overlays are applied to extend the service life of an existing bridge deck by providing a new wearing surface and reducing the ingress of moisture and chlorides into the deck, which commonly corrode the steel reinforcement present in the deck. Overlays not only provide waterproofing and protection from chlorides, but, in other cases, they are installed to strengthen and/or stiffen a deteriorated bridge deck. UHPC-class materials offer several properties that make them advantageous for bridge deck overlays:

- Very low permeability and very good resistance to freeze-thaw damage: UHPC significantly reduces the potential for ingress of contaminates as well as freeze-thaw damage compared with conventional overlay and partial-depth replacement materials, thus minimizing the maintenance cost and increasing the bridge deck lifespan.

- Good abrasion resistance: UHPC reduces the potential for wheel-path abrasion on the ride surface of the bridge deck.

- High strength and stiffness: A thin layer of UHPC could provide both enhanced durability and increased flexural strength with minimal added dead load.

- High bond strength: UHPC has a high bond strength and can act compositely with existing concrete surfaces, if those surfaces are properly prepared (De la Varga, Haber, and Graybeal 2017; Aaleti and Sritharan 2017, 2019).

- Cost-effective lifecycle: UHPC offers a cost-effective alternative to deck replacement and some other deck rehabilitation options. UHPC may cost more up front, but it will last longer and require less maintenance than other solutions.

Example Projects

UHPC bridge deck overlays have been installed on more than 150 bridges worldwide as of 2020. UHPC application as a bridge deck overlay originated in Switzerland. As such, the majority of completed projects are in Switzerland (Brüwiler and Denarié 2013; École Polytechnique Fédérale de Lausanne n.d.). As of 2020, 17 UHPC bridge deck overlays have been installed in the United States (FHWA 2021).

Bridge Over the Morge River

One of the earliest UHPC bridge deck overlays was installed in Châteauneuf-Conthey, Switzerland in 2004. This bridge spans the Morge River and is shown in figure 13. The primary objective of the UHPC overlay was to waterproof the deck and arrest active deterioration of concrete and steel caused by the ingress of water and chlorides. The bridge deck surface was prepared by hydrodemolition. This project employed a 1.18-inch-thick UHPC overlay. An asphalt wearing surface was installed atop the UHPC overlay. A waterproofing membrane was not used between the UHPC overlay and the asphalt topping.

Figure 13. Photo. Bridge over the Morge River in Châteauneuf-Conthey, Switzerland.

The UHPC overlay performance was evaluated after approximately 10 years of service. The UHPC overlay was found to be performing as anticipated: elevated chloride levels were only found within the first 0.1-inch of the UHPC overlay. Figure 14-A shows a photo of the bridge deck soffit shortly after the overlay was installed. Spalled concrete, exposed and corroded reinforcement, and efflorescence are observed. Figure 14-B shows a photo taken after 9.5 years of service. As shown, the condition of the deck soffit remains unchanged. That is, deterioration did not appear to progress, and there were no indications of additional efflorescence. In Switzerland, damaged deck soffits are not commonly left unrepaired. In this case, the soffit was left in an unrepaired state to provide a visual comparison over time (Denarié 2015).

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A. July 2005. B. March 2014.

Figure 14. Photos. Soffit of the bridge over the Morge River.

Chillon Viaduct

The Chillon Viaduct, located near Montreux, Switzerland, was rehabilitated using a UHPC overlay in 2015 (figure 15). This major highway structure is composed of twin single-cell segmental concrete box girder structures, each 1.5 mi long, with a total deck surface of about 580,000 square ft (Brühwiler et al. 2015). The viaduct structures, built in 1969, were deteriorating from alkali aggregate reactivity as well as traffic volumes and truck weights that exceeded the original design assumptions. UHPC was selected to provide enhanced durability and waterproofing and to increase the capacity of a relatively thin deck (7.1 inches), which was integral to the box girders. The UHPC overlay was designed to be 1.5–2 inches thick, depending on the location on the deck. To provide additional capacity, the UHPC overlay was reinforced with No. 5 bars. Bars were placed transverse to the direction of traffic and longitudinal over pier regions (figure 16). The reinforced UHPC overlay increased the transverse moment capacity of the deck by 73 percent in negative bending and by 33 percent in positive bending.

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Figure 15. Photo. Chillon Viaduct during UHPC deck overlay construction in 2015.

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Figure 16. Photo. Placing UHPC on the deck of the Chillon Viaduct in the vicinity of a pier.

Laporte Road Bridge Over Mud Creek

The first UHPC bridge deck overlay constructed in the United States was part of a demonstration project in Iowa and was completed in 2016 (Sritharan et al. 2018). The UHPC overlay was installed on a three-span, cast-in-place concrete slab bridge carrying Laporte Road over Mud Creek in Buchanan County, IA (figure 17). The primary objective of the project was to evaluate the feasibility of installing UHPC deck overlays in the field with the anticipation of accomplishing the following tasks: repairing surface deterioration, waterproofing the bridge deck surface, and providing a new riding surface. Six months after installation, the bond between the UHPC overlay and the substrate concrete was assessed in a field study conducted by FHWA researchers. Based on observations and data collected, the team concluded that the bond between the UHPC overlay and the existing concrete bridge deck was sound, and the installation successful. See Haber, Munoz, and Graybeal (2017) for additional information.

Source: FHWA.

Figure 17. Photo. Placing UHPC on the Laporte Road Bridge in 2016.

Pilot Projects on Long-Span Bridges in the United States

In 2020, UHPC overlays were installed on three long-span bridges in the United States. These projects were carried out as pilot projects for which UHPC overlays were installed on large areas of bridge deck. The bridges included in this list are as follows:

- Commodore Barry Bridge: This cantilever truss bridge opened in 1974, connects Pennsylvania and New Jersey, and is owned by the Delaware River Port Authority (DRPA). The UHPC overlay pilot project included UHPC installation on a deck truss span and a girder span. Full-scale UHPC application could save the DRPA more than $200 million in capital costs and extend the life of the bridge deck by 30 years or more (Summers 2021). Figure 18 shows a construction crew preparing the bridge deck of the deck truss span with hydrodemolition in preparation for installing the UHPC overlay.

- Delaware Memorial Bridge, first structure: This suspension bridge opened in 1951. It connects Delaware and New Jersey and is owned by the Delaware River and Bay Authority (DRBA). The UHPC overlay pilot project included UHPC installation on girder, truss, and suspension spans. Full-scale application could save the DRBA more than $60 million compared with a full bridge deck replacement (FHWA 2021). A photo of this project is shown in figure 19.

- Claiborne Pell Bridge: This suspension bridge opened in 1969 and is owned by the Rhode Island Turnpike and Bridge Authority. The UHPC overlay pilot project included UHPC installation on the suspension spans. Installation was done by two different UHPC suppliers. A photo of this project is shown in figure 20.

Source: FHWA.

Figure 18. Photo. Hydrodemolition of the bridge deck of the Commodore Barry Bridge before installation of a UHPC overlay.

© 2020 Delaware River and Bay Authority.

Figure 19. Photo. UHPC overlay on the Delaware Memorial Bridge before grinding.

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Figure 20. Photo. UHPC overlay installation on the Claiborne Pell Bridge.

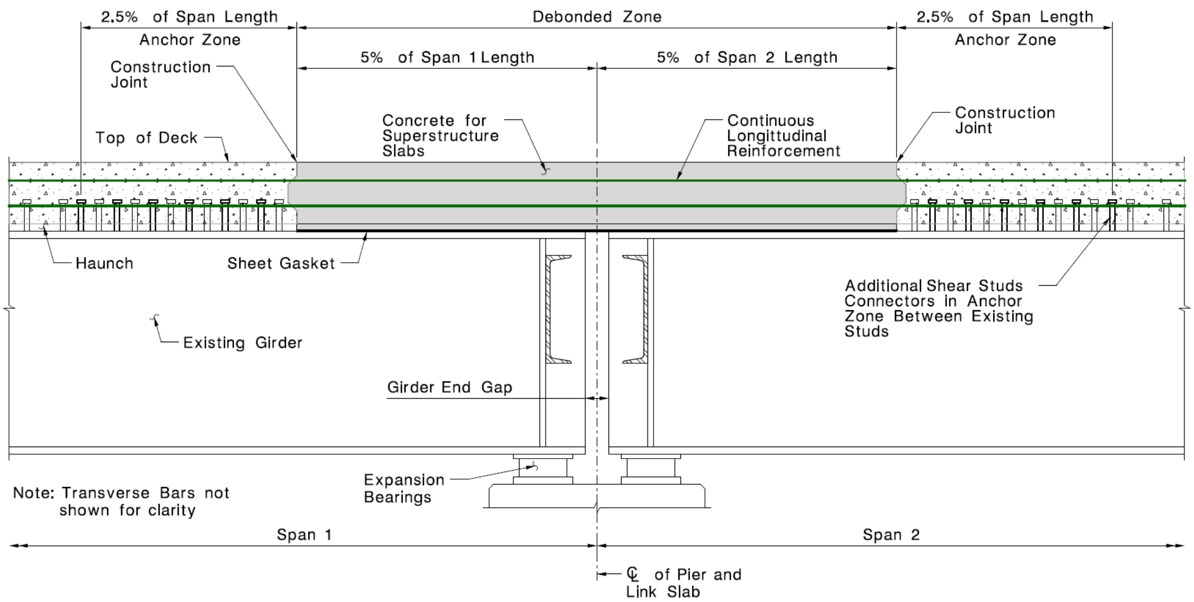
EXPANSION JOINT REPLACEMENT WITH UHPC LINK SLABS

Background

In simple span bridge construction, deck joints are commonly provided at the ends of the simple spans. These joints accommodate girder end rotations and deck translation caused by live loads, temperature change, and concrete creep and shrinkage. A common issue with simple span construction is sealing deck joints. Deck joint seals often fail prematurely, allowing water and deicing chemicals to leak through. This leakage causes deterioration of girder ends, bearings, and substructure elements (Thorkildsen and Greenman Pedersen Inc. 2020). Additionally, joint seal materials, armoring angles, and concrete headers frequently become dislodged by heavy vehicles such as snowplows and become scattered across the roadway, creating hazardous conditions for motorists. One solution to deck joints issues is to elimination the joints altogether and replace them with link slabs. A link slab is a structural slablike element, traditionally composed of reinforced concrete, that is placed between adjacent simple spans and ties into the existing bridge deck. The link slab concept was first proposed in the early 1980s (Zuk 1982) and has since been deployed by at least 30 percent of U.S. DOTs (Haikal et al. 2019).

Design Concepts Related to Link Slabs

Link slabs are designed to accommodate girder end rotations and superstructure deformations without introducing moment continuity between adjacent spans. As noted by Caner and Zia (1998), the stiffness of the link slab is much lower than that of the composite deck-girder system. As such, link slabs can be designed and detailed to maintain a bridge’s simple span behavior for analysis purposes even after the ends of the adjacent bridge decks are linked together. Traditionally, link slabs have been composed of reinforced concrete. To accommodate deformation at the beam ends, well-distributed cracking is expected to occur in the concrete. As such, early research conducted by Caner and Zia (1998) recommended debonding the link slab from the remainder of the superstructure to allow for this behavior to occur. The recommended debonded length was equal to 5 percent of the total span length adjacent to the link slab; the length of span 1 was added to the length of span 2. Lastly, many States who deploy link slabs have instituted limits on crack widths to reduce the potential of leakage and moisture ingress (Thorkildsen and Greenman Pedersen Inc. 2020). Figure 21 depicts a modern link slab designed using conventional reinforced concrete.

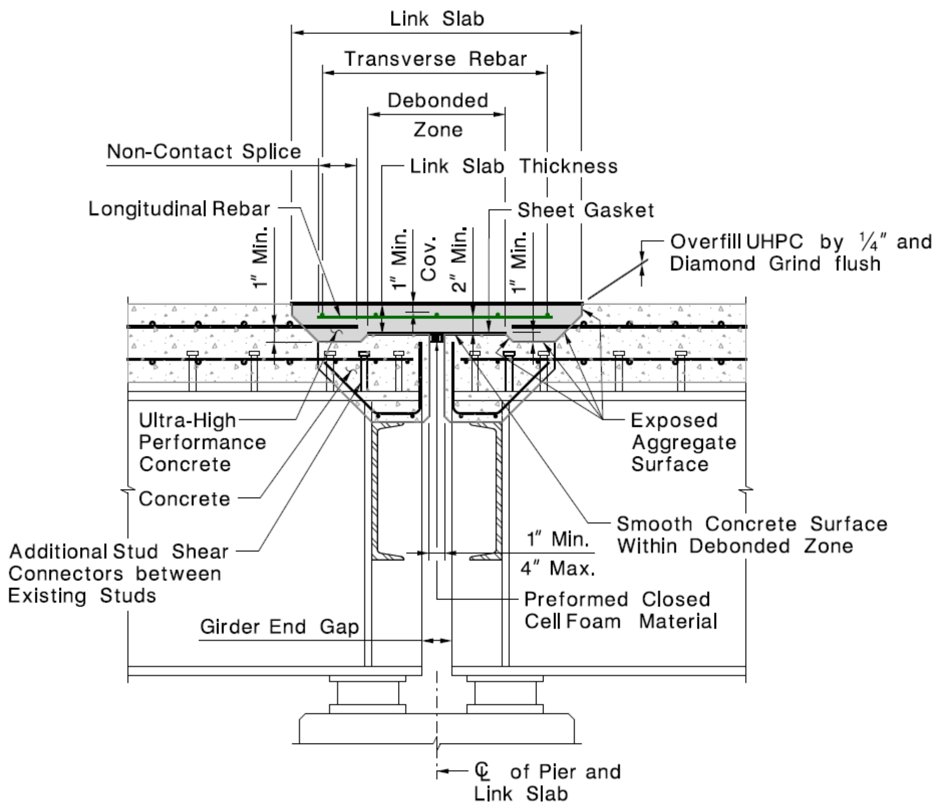


Source: FHWA.

Figure 21. Illustration. Conventional link slab section.

UHPC Link Slabs

The first U.S. bridge to use a UHPC link slab was built by New York State DOT (NYSDOT) in 2013 in Owego, NY. The bridge carries Route 926G over Route 17. This structure is composed of two simply supported steel, multigirder spans and carries two lanes of traffic. The deck was replaced using full-depth precast deck panels connected with UHPC. At the same time, the deck joint over the pier was eliminated by using a UHPC link slab. To date, NYSDOT has completed the most UHPC link slabs in the United States and has institutionalized the technology as they have developed standard design details, sample calculations, and specifications, making the deployment of this technology widespread throughout the State. Figure 22 shows a common design detail for a UHPC link slab in the State of New York. As of 2020, six different State DOTs have deployed this technology, and UHPC link slabs have been installed on at least 35 bridges in the United States. UHPC offers several advantageous for link slabs, such as:



Source: FHWA.

Figure 22. Illustration. UHPC link slab details.

- Optimized link slab size and geometry: UHPC link slabs might be significantly shorter (50 percent or more) than those composed of conventional concrete because of UHPC's tensile ductility. UHPC-class materials have a high tensile strain capacity. As such, the deformation demand imposed on the UHPC link slab by the adjacent spans can be accommodated by a relatively short length of slab. For example, on a given bridge, a UHPC link slab may only need to be 2–4 ft in length, whereas a conventional concrete link slab may need to be 6–9 ft in length. Furthermore, UHPC link slabs can be relatively thin with reduced supplemental reinforcement compared with conventional concrete, thus, minimizing the moment continuity over the pier location and maintaining simply supported behavior.

- Simplified construction: Since UHPC link slabs can be designed to be relatively small, they require less onsite labor and material volume, which expedites and simplifies construction. This shortened construction period reduces the impact on the traveling public because construction windows can be shortened. Figure 23 shows the construction sequence of a UHPC link slab in New York State that is replacing an existing expansion joint.

- Enhanced durability: Lastly, UHPC link slabs are inherently more durable than those constructed with conventional concretes, due to the durability properties of UHPC. Furthermore, the cracks that form in UHPC are designed to be very thin compared with conventional concrete. Thus, the risk of leakage from moisture and deicing salt from the deck surface to the beam ends and substructure is reduced.

A. Removal of existing concrete.

B. Placement of UHPC.

C. Finished UHPC link slab.

Figure 23. Photos. Installation of a UHPC link slab in New York State.

STEEL GIRDER END REPAIRS USING UHPC

Background

Deterioration of steel girder ends is a common issue encountered by bridge owners. This deterioration, usually in the form of corrosion of the steel (figure 24), is due to leakage of water and deicing agents from expansion joints. The corrosion leads to section loss of the webs, flanges, and stiffeners at the beam ends. This loss reduces the shear and bearing capacity of the girder. To keep bridges operating safely, these deteriorated regions require repair when the section loss becomes significant. Conventional repairs include incapsulating the entire end of the superstructure with a reinforced concrete integral diaphragm, or replacing the damaged steel section with new steel, or, in extreme cases, replacing the entire girder. In some cases, the superstructure or bridge deck may require jacking and/or lane closures to install these solutions. This situation makes completing these repairs challenging at times.

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Figure 24. Photo. Steel beam end corrosion damage.

UHPC offers an innovative alternative to the traditional repair solutions to restore the shear and bearing capacity of deteriorated steel girder ends. This alternative repair is illustrated in figure 25. The UHPC-based repair employs thin, full- or partial-height panels of UHPC cast against the girder web, encasing the corroded portions of steel. The UHPC panels are anchored to the girder by headed shear connectors welded to the intact, noncorroded portions of the web. Alternative shear connectors have also been investigated in lieu of headed studs and include threaded rods and perforated web connections (Kruszewski, Wille, and Zaghi 2018). This design can effectively create a new load path that bypasses the deteriorated regions of steel and restores the lost bearing and shear capacity of the beam. Additionally, the UHPC panel protects the underlying steel from future section loss (Kruszewski 2018). Kruszewski noted that encasing both sides of the beam end in UHPC reduces the inspectability of the beam end, although the use of UHPC is expected to mitigate future web damage. The benefits of this repair method are summarized as follows:

All images: © 2020 Arash Zaghi, University of Connecticut.

Figure 25. Photos. Installation of a UHPC beam end repair.

- Enhanced durability: UHPC's low permeability and high freeze-thaw resistance reduce further corrosion and the potential for additional section loss in the girder. Conventional repair solutions cannot achieve these results.

- Constructability: UHPC repairs can be performed in short time windows and may reduce the need to jack the superstructure. Additionally, this repair is easily constructed using standard construction procedures, such as standard stud welding and carpentry for concrete formwork.

- Minimal impact to users: The impact on end users is minimized by reducing onsite mobilization and repair time, thereby also reducing the need for lane closures. This repair strategy can also be performed periodically during off-peak hours, making it ideal for projects with short time windows and without disruption to traffic.

- Repair versatility: This repair strategy can be easily adapted to different field conditions, such as bridges with complex geometries and projects with limited access to the repair area. These conditions are typical in bridges with varying degrees of skew.

- Low maintenance requirements: This repair method could potentially be maintenance free or require minimal maintenance if it is properly detailed and installed.

This repair method was developed over the last decade through successive research projects funded by Connecticut DOT (CTDOT) and performed by researchers at the University of Connecticut. The goal of these successive projects was to develop, evaluate, and validate a UHPC-based rehabilitation strategy for corroded steel girder ends. The research included small-scale testing of different shear connectors, large- and full-scale testing of repaired steel girders, computational modeling and parametric study of the repair strategy, and technical assistance and monitoring of CTDOT's first implementation of the repair on an actual bridge. The reader is referred to the works by Zaghi et al. (2017a), Kruszewski, Wille, and Zaghi (2018), Zaghi et al. (2017b), and Hain et al. (2019) for additional details.

Examples of UHPC Steel Girder End Repair

Following are several known deployments of steel girder end repairs using UHPC in the United States:

- Rhode Island DOT (RIDOT): RIDOT completed its first beam end repair with UHPC in 2018 in Providence, RI, as an emergency repair of the bridges at the interchange of Route 6 and Route 10. This project included UHPC beam end repairs on 18 beam ends. An example is shown in figure 26. In addition to having UHPC throughout the entire web height at the bearing, a 6-ft UHPC repair was installed along the bottom flange of the beams to rehabilitate extensive corrosion of the bottom flange.

- CTDOT: CTDOT completed its first beam end repair with UHPC in 2018 in New Haven, CT, where Interstate 91 crosses three sets of railway tracks (Hain et al. 2019). This project had extensive constraints, as work was only permitted in a single 4-hour timeframe each day. Additionally, the superstructure was composed of rolled wide-flange beams with depths ranging from 33 to 36 inches. Also, because of the rail alignments, each of the piers intersected the superstructure at a different skew angle. The project involved the repair of 42 beam ends. Because of the aforementioned superstructure geometric conditions, each beam end was unique and would have resulted in well over 100 different shop drawings, if a conventional bolted or welded repair option were pursued, due to varying plate thicknesses, widths, and skews. The photos shown in figure 25 are from this project.

- St. Clair County, MI: The St. Clair County Road Commission completed its first beam end repair with UHPC in 2018 on Masters Road-Belle River Bridge. This bridge was built in the 1930s in St. Clair County, MI. The bridge had experienced expansion joint failure and subsequent deterioration. The county chose to rehabilitate the bridge instead of replacing it due to its historical value. UHPC was selected for the repair of the beam ends. Welding headed shear connectors was not an option due to the poor condition of the beams. Instead, reinforcing bars were used, which were passed through holes drilled in the webs of all the beams. A nonproprietary UHPC mix design was used.

- Texas DOT (TxDOT): TxDOT completed its first beam end repair with UHPC in 2020 on the Sidney Sherman Bridge, which carries Interstate 610 over the Houston Ship Channel in Houston, TX (figure 27-A). Built in 1969, the bridge had begun to show significant corrosion damage to the girder ends and diaphragms, as well as rocker bearing deterioration. UHPC was selected due to its durability, high strength, and ease of installation compared with other solutions. Figure 27-B shows one of the completed UHPC repairs.

Source: FHWA.

Figure 26. Photo. UHPC beam end repair on the Route 6/10 Interchange Bridge.

Source: Texas DOT.

A. Elevation view of Sidney Sherman Bridge.

Source: Texas DOT.

B. Completed beam end repair with UHPC.

Figure 27. Photos. UHPC beam end repair on the Sidney Sherman Bridge.

CHAPTER 4. DESIGN OF UHPC BRIDGE P&R APPLICATIONS

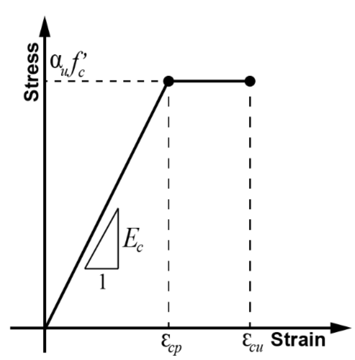
This section provides both general and specific design recommendations for UHPC-based P&R applications. The general recommendations cover items somewhat universal to construction with UHPC and would be applicable for most P&R applications. Specific design recommendations are provided for three promising UHPC P&R applications: bridge deck overlays, link slabs, and steel beam end repair. The recommendations included herein are based on materials and structural engineering research that has been conducted on UHPC-class materials by FHWA and other entities, as well as lessons learned from field deployments and in-service applications. Recommendations are presented in tables in a dual-column format. Similar to many structural design codes, recommendations are provided in the left column, and the corresponding commentary is provided in the right column.

STRESS-STRAIN RELATIONSHIPS FOR DESIGN

The following subsection presents the uniaxial stress-strain relationships recommended for design with UHPC. These relationships have been developed through considerable research at FHWA's TFHRC. These research findings and additional background on how these relations were developed are given in the article by El-Helou, Haber, and Graybeal (2022).

Compression Relationship

The recommended compression stress-strain relation is shown in figure 28. This relationship should be treated as linearly elastic until a stress of $\alpha_u f'_c$ is reached. Here, α_u is the reduction factor, which is taken as 0.85, at a maximum. After the effective compressive strength, $\alpha_u f'_c$, is reached, the compressive resistance is constant until the ultimate compression strain, ε_{cu}, is reached. The compression parameters—modulus of elasticity, E_c; compression strength, f'_c; and ε_{cu}—and Poisson's ratio, v, shall be obtained from cylindrical specimens tested according to ASTM C1856/C1856M (ASTM 2017a). In lieu of experimental test data, the modulus of elasticity can be determined by equation 1 (described in the *General Recommendations, Modulus of Elasticity* subsection), Poisson's ratio may be taken as 0.15, and the ε_{cu} may be taken as 0.0035.

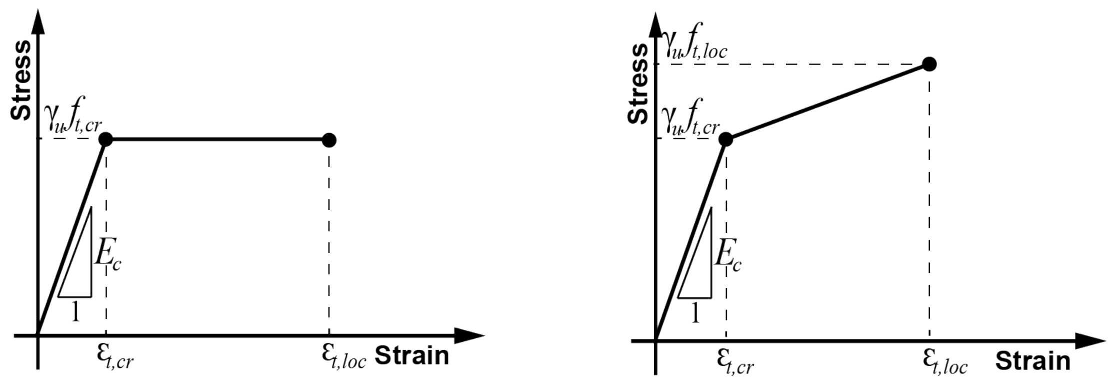


Source: FHWA.
ε_{cp} = transition point between the elastic and plastic portions of the curve.

Figure 28. Image. Recommended compressive stress-strain relationship.

Tension Relationships

The recommended tension stress-strain relations are shown in figure 29. These relationships should be treated as linearly elastic until a stress of $\gamma_u f_{t,cr}$ is reached, where γ_u is a reduction factor and $f_{t,cr}$ is the effective cracking strength. Other parameters include the E_c; the effective cracking strain, $\varepsilon_{t,cr}$; the localization stress, $f_{t,loc}$; and the localization strain, $\varepsilon_{t,loc}$. Herein, "localization" is defined as the point at which tensile deformation in UHPC begins to accumulate into a single dominant crack. After this point, tensile stress continuously decreases with increasing strain or permanently drops below the value of the effective cracking stress.



Source: FHWA.

A. Stress plateau response.

Source: FHWA.

B. Stain-hardening response.

Figure 29. Images. Recommended tension stress-strain relationships.

These parameters shall be obtained from prismatic specimens tested in uniaxial tension according to AASHTO T397 (AASHTO 2022). Additional details regarding how these relationships were developed and how to determine individual parameters are found in the work by El-Helou, Haber, and Graybeal (2022). The elasto-plastic model (figure 29-A) should be used when average tension test results, determined according to AASHTO T397, demonstrate that $1.2f_{t,cr} < f_{t,loc}$. The bilinear model (figure 29-B) may be used when average tension test results demonstrate that $1.2f_{t,cr} \geq f_{t,loc}$. For the elasto-plastic model, $f_{t,loc}$ should not be taken to be less than $f_{t,cr}$. The reduction factor γ accounts for variability in tensile response and can be taken to be 0.85 in lieu of statistical data from the UHPC mixture of interest.

MINIMUM PROPERTIES OF UHPC

The recommendations presented herein are for current UHPC-class materials, which are cement composite materials composed of an optimized gradation of granular constituents, a water-to-cementitious materials ratio of less than 0.28, and a high percentage of discontinuous internal steel fiber reinforcement, which ensures tensile ductility. The design recommendations in the following subsections are predicated on the minimum mechanical properties of UHPC listed in table 2.

Table 2. Minimum mechanical properties of mature-age UHPC.

Property	Variable Symbol	Test Method	Minimum Value
Compressive strength	f'_c	ASTM C39 and ASTM C1856 (ASTM 2017a, 2020a)	18 ksi
Effective cracking strength	$f_{t,cr}$	AASHTO T397 (AASHTO 2022)	0.75 ksi
Localization stress	$f_{t,loc}$	AASHTO T397 (AASHTO 2022)	$f_{t,loc} \geq f_{t,cr}$
Localization strain in direct tension	$\varepsilon_{t,loc}$	AASHTO T397 (AASHTO 2022)	0.0025
Steel fiber reinforcement	V_f	Not applicable	2.0 percent (by volume)*

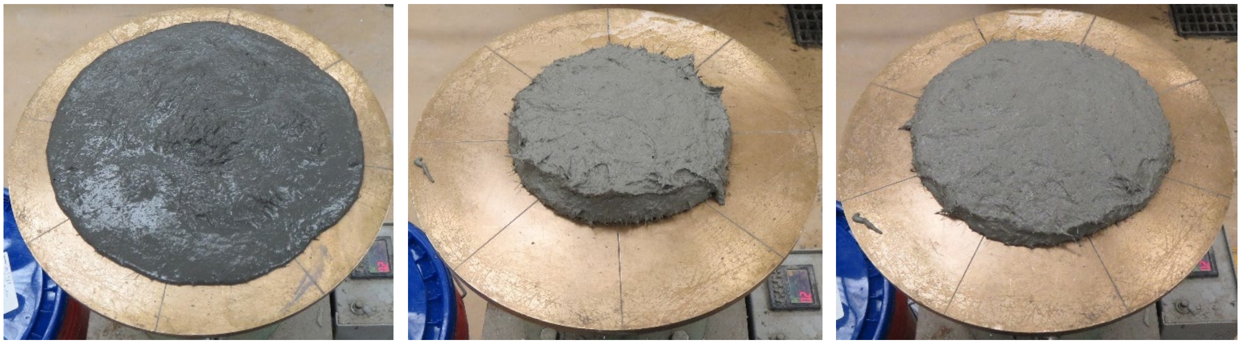
*A higher fiber content may be used to increase the tensile strength as needed to meet the demands of the particular UHPC application.

RHEOLOGY OF UHPC FOR FIELD APPLICATIONS

The rheology or "workability" of UHPC-class materials typically used in field applications falls into one of two categories:

- Self-leveling: Self-leveling UHPCs are highly flowable and do not require mechanical vibration to consolidate. That is, these UHPCs are formulated to flow under the force of gravity. When evaluated using a static flow table test, per ASTM C1856, self-leveling UHPCs commonly exhibit spread diameters between 6 and 10 inches (ASTM 2017a). Figure 30-A shows a self-leveling UHPC mixture after completion of a static flow table test. Self-leveling UHPCs are commonly used for applications that require significant flow capabilities, such as headers, pier or column encasement, link slabs, connection repairs, or beam end repairs.

- Thixotropic: Thixotropic UHPCs exhibit thixotropic material behavior and require internal or external vibration for consolidation. Thixotropy is a time-dependent shear thinning property of a non-Newtonian fluid, which causes a material to remain solidlike under static conditions and to flow when agitated or sheared. Figure 30-B shows a thixotropic UHPC mixture after completion of a static flow table test, and figure 30-C shows the same mixture after a dynamic flow table test (20 drops). Thixotropic UHPCs are mostly used for bridge deck overlays. Bridge decks are not level, and as such thixotropic mixtures allow UHPC to be placed on sloped bridge decks, up to a maximum grade of 10 percent, while maintaining the required profile. The UHPC supplier will typically vary the UHPC thixotropic flow properties according to the slope of the deck, so the spread will be greater for small slopes and less for larger slopes.



Source: FHWA.

A. Self-leveling UHPC after static testing.

Source: FHWA.

B. Thixotropic UHPC after static testing.

Source: FHWA.

C. Thixotropic UHPC after dynamic testing.

Figure 30. Photos. Flow table testing of self-leveling and thixotropic UHPC formulations.

GENERAL RECOMMENDATIONS

Unit Weight

Recommendations	Commentary
The unit weight of UHPC inclusive of the steel fiber reinforcement may be taken as 155 lb/ft^3.	This value most closely represents the unit weight of UHPC with 2 percent (by volume) steel fiber reinforcement. Lesser or greater percentages of steel fiber reinforcement necessarily decrease or increase the unit weight, respectively. Inclusion of non-steel fiber reinforcement also affects the unit weight.

Chloride Ion Diffusion Coefficient

Recommendations	Commentary
The diffusion coefficient of UHPC may be taken as 2.0×10^{-10} inches^2/s.	This value is based on research completed on UHPC cementitious matrices with cementitious materials contents greater than 1,500 lb/yd^3, no aggregates larger than a fine sand with an average diameter of 0.02 inches, and water-to-cementitious materials ratios less than 0.25 (Association Française de Génie Civil 2013; Thomas et al. 2012; Kono et al. 2013; Piérard, Dooms, and Cauberg 2013).

Coefficient of Thermal Expansion

Recommendations	Commentary
In the absence of more precise data, the coefficient of thermal expansion of UHPC may be taken as 7.0×10^{-6} inches/inch/°F.	The coefficient of thermal expansion value depends on the UHPC material constituents and should be based on laboratory tests and data provided by the UHPC material supplier. The coefficient of thermal expansion value shown herein is based on the experiment by Mohebbi, Graybeal, and Haber (2022)

Modulus of Elasticity

Recommendations	Commentary
The modulus of elasticity of UHPC may be calculated as follows: $$E_c = 2{,}500\,(f'_c)^{1/3} \text{ (in ksi)} \quad (1)$$	This equation is based on research results obtained from testing steel fiber-reinforced UHPCs and is a good approximation for UHPCs with compressive strengths between 14 and 29 ksi (El-Helou, Haber, and Graybeal 2022).

Bond Strength to Existing Concrete or UHPC

Recommendations	Commentary
Concrete substrate surfaces should be roughened to enhance interface bond strength. Prepared surfaces should have preferably both macro- and microtexture. The preparation methods should be selected to minimize microcracking in concrete, otherwise known as "bruising."	The interface tensile bond strength can range from zero to the tensile strength of the substrate concrete. Laboratory research has demonstrated that UHPC can exhibit interface bond strengths between 0.35 and 0.6 ksi when the substrate concrete has been prepared to exhibit both macro- and microtexture (i.e., exposed aggregate) (De la Varga, Haber, and Graybeal 2017). Removing the surface cement paste to provide microtexture is more important than providing macrotexture for tensile bond strength, although macrotexture is important for interface shear strength (De la Varga, Haber, and Graybeal 2017). Providing macrotexture without microtexture, such as occurs with a form liner, is not recommended for bond-critical applications such as used to prevent moisture intrusion. UHPC will not bond well to smooth or dirty surfaces.
Hardened UHPC substrate surfaces should be roughened to include microtexture and	Fresh UHPC can bond to hardened UHPC under the right conditions. Fibers can be

Recommendations	Commentary
possibly macrotexture. The projection of fiber reinforcement from the substrate UHPC into the interface region is considered beneficial.	exposed using set retarders on the formwork, which will delay the hydration reaction of the fresh UHPC contacting the form. After form removal, the nonhydrated surface paste can be washed from the hardened UHPC with water. Hardened UHPC can be roughened, and the fibers exposed by sand blasting.
Existing concrete surface should be prewetted with water to a saturated surface dry (SSD) condition.	Prewetting concrete has been demonstrated to increase the bond strength between UHPC and the existing concrete. This procedure typically requires a minimum of 6 hours or more of continuous wetting. Bonding agents can also be considered, but information is lacking on the long-term performance of bonding agents.

Development Length of Reinforcement

Recommendations	Commentary
The recommendations herein may be used for deformed steel reinforcement sizes No. 8 and smaller embedded in field-cast UHPC with a compressive strength of at least 14 ksi at the time of loading.	A compressive strength of 14 ksi is defined here to facilitate the use of UHPC in accelerated construction when early application of construction loads is advantageous. The final compressive strength of UHPC is normally significantly greater than 14 ksi.
The embedment length of deformed steel reinforcement in UHPC should be equal to or greater than the development length, l_d. When cover is $\geq 3d_b$: • $l_d \geq 8d_b$ for reinforcing bars with yield strength $f_y \leq 75$ ksi. • $l_d \geq 10d_b$ for reinforcing bars with yield strength 75 ksi $< f_y \leq 100$ ksi. Where: d_b = diameter of reinforcing bar. f_y = yield strength of reinforcing bar.	Research has demonstrated that deformed steel reinforcement can be developed within comparatively short embedment lengths (Yuan and Graybeal 2014). An embedment length of $8d_b$ is sufficient for most common reinforcement configurations, including the use of epoxy-coated reinforcement. Increased confinement of the bar and increased mechanical properties of UHPC can allow even shorter l_d values. Bars with higher f_y values require an increase in the development length.
When $2d_b \leq$ cover $< 3d_b$, increase the minimum l_d by $2d_b$.	A decrease in the cover results in reduced confinement of the bar and thus an increase in the required development length.
For concrete bridge deck applications, the	For No. 5 bars embedded in UHPC with

Recommendations	Commentary
embedment length of a No. 5 deformed steel reinforcement bar can be taken as: When cover is ≥ 1.25 inches: • $l_d \geq 8d_b$ for reinforcing bars with yield strength $f_y \leq 75$ ksi. When 1.0 inch ≤ cover <1.25 inches: • $l_d \geq 10d_b$ for reinforcing bars with yield strength $f_y \leq 75$ ksi.	cover as small as 1 inch, research has demonstrated that they sustain stress levels comparable with those expected at the ultimate limit state.

Lap Splices of Reinforcement

Recommendations	Commentary
For lap splices of straight lengths of deformed steel reinforcement, the lap splice length, l_s, should be at least $0.75l_d$. Clear spacing to the nearest lap-spliced bar should be $\leq l_s$. Clear spacing between adjacent bars should also meet the clear spacing requirement defined in *Minimum Cover and Spacing of Reinforcing Bars* subsection.	Research by Yuan and Graybeal (2014) demonstrated that passive reinforcement can develop stresses in excess of 100 ksi when l_d is embedded into a UHPC connection and spliced with adjacent bars with a lap of $0.75l_d$.

Minimum Cover and Spacing of Reinforcing Bars

Recommendations	Commentary
The minimum cover of UHPC around reinforcements or other embedments, and the clear spacing between adjacent reinforcements or other embedments, should not be less than the greater of 1.5 times the length of the longest type of fiber reinforcement included in the UHPC or 0.75 inch, unless adequate fiber distribution is otherwise demonstrated for a specific application.	This recommendation stipulates a geometry that allows for sufficient fiber passing through constricted spaces during casting. Restrictions to UHPC flow during casting can result in undesirable fiber distribution effects. This provision could be relaxed in specific applications, but adequate fiber distribution would need to be demonstrated first.

Formwork and Traffic Vibration Mitigation

Recommendations	Commentary
Formwork should be designed and installed to be watertight and must withstand hydrostatic pressures from UHPC and buoyancy forces on any top forms.	UHPC can be highly fluid and may not self-cauterize leaks in the formwork that normal concrete may be capable of doing. UHPC has a higher unit weight than normal concrete, and formwork has to be designed

Recommendations	Commentary
	considering the higher hydrostatic pressure.
	Watertightness can be established via mock-ups or in the actual forms. If watertightness is established in the actual forms, excess water should be removed before UHPC placement.
Formwork surfaces in contact with UHPC should have a nonabsorbent finish so they will not pull moisture out of the UHPC.	Oiled plywood will reduce the amount of moisture absorption compared with untreated plywood, but it may still result in some moisture absorption. Options for truly nonabsorbent formwork include steel, plywood products with a resin coating, or plywood forms wrapped with plastic sheeting.
External vibration from traffic should be greatly reduced or eliminated during the time period after casting and before the UHPC has developed an acceptable level of mechanical properties. Maintain vibration mitigation until the UHPC has achieved a compressive strength of 14 ksi.	Excessive vibration before the UHPC sets can result in the segregation of fibers and can affect the bond between embedded connectors or reinforcement and UHPC. Fibers settlement can produce mechanical property variability across a volume of cast UHPC.
Maintain formwork until the UHPC has achieved a compressive strength of 14 ksi.	

Mixing

Recommendations	Commentary
UHPC mixtures should be batched and mixed according to developer or manufacturer recommendations, with consideration given to the ambient environment.	UHPC performance is sensitive to mixing deviations. Addition of water or chemical admixtures above or below the developer- or manufacturer-established ranges can be detrimental to the early and long-term performance of the material.
	In hot weather, UHPC is commonly batched with a flow at the higher end of the flow range to allow for more working time. In cold weather, batching UHPC with the lower end of the flow range is preferable to reduce any potential for fiber settlement from construction vibrations during longer setting times.
Stockpiled materials and mix water should be stored at reduced temperatures.	Environmental conditions can affect the properties of UHPC before and during the

Recommendations	Commentary
	mixing operations. The temperature of UHPC increases during mixing, and some mix water is typically lost to evaporation. Best practices call for mixing operations to occur under cool temperatures and away from direct exposure to sun and wind, if possible.
A mixer that is be capable of dispersing the liquids and fibers uniformly within both the powder and fluid matrix should be used to mix UHPC.	UHPC can be mixed in most concrete or grout mixers. Both tow-behind pan mixers and conventional concrete ready-mix trucks have been used to mix UHPC. As a general rule, the maximum volume of UHPC that can be mixed in a conventional mixer is between approximately one-third and two-thirds of the volume of conventional concrete or grout that can be mixed.
The temperature of the UHPC at the conclusion of mixing should be kept between 40 and 80 °F, unless otherwise approved by the owner.	UHPC gains heat during the mixing process. Typically, UHPC mixtures that exceed 80 °F exhibit diminished flowability, decreasing the time available for placement and increasing the likelihood of surface dehydration. Ice cubes are a viable replacement for some or all of the mix water during warm or hot weather conditions.

Placement and Consolidation

Recommendations	Commentary
Fresh UHPC should be transported, placed, and covered as quickly as possible to minimize the potential for reduced flowability and avoid evaporation (loss) of mix water from exposed surfaces.	Methods to estimate conventional concrete surface dehydration rates are available and may be applicable to UHPC-class materials (ACI 2014).
Highly flowable, self-leveling UHPCs should not be externally or internally vibrated.	Vibration of these type of UHPC can cause segregation and settlement of fiber reinforcement. Rodding is acceptable and can be used in situations where two successive pours meet. Tapping with a hammer on top forms can help remove entrapped air and provide an audible QC check.
Thixotropic UHPCs should be vibrated as necessary such that consolidation is achieved. Constituent segregation should be avoided.	

Curing and Strength Gain

Recommendations	Commentary
UHPC should be protected from freezing temperatures until it attains the specified minimum compressive strength of 14 ksi, unless otherwise specified.	Similar to conventional concretes, UHPC is susceptible to the deleterious effects of freezing temperatures before gaining appropriate strength. Laboratory studies have demonstrated that UHPCs commonly exhibit their full mechanical and durability properties once a compressive strength of 14 ksi is reached (Haber et al. 2018). Afterwards, UHPC may be exposed to the environment without experiencing any deleterious effects. Although cooler temperatures are beneficial for mixing and placing UHPC, warmer temperatures are beneficial to initiate the initial set, rapid curing, and improved mechanical properties (Esmaeili and Kasaei 2016; Sbia et al. 2017). UHPC overlay projects, where reinforcing bar development and lap splice capacity are not a concern, have permitted the removal of moisture protection and the application of construction live loading at strengths as low as 11 ksi (76 MPa) with no apparent detrimental effects.
Exposure to the external environment, such as direct sunlight or wind, should be limited before UHPC attains the specified minimum compressive strength to prevent surface dehydration.	Excessive surface dehydration can lead to plastic and/or drying shrinkage cracking. This concern is more common as it relates to bridge deck overlays and other applications with large, potentially exposed surface areas.

SPECIFIC RECOMMENDATION: BRIDGE DECK OVERLAYS

This subsection provides recommendations for bridge deck overlays using UHPCs that meet the requirements listed in table 2. These recommendations are based on best practices in the United States and projects complete in Switzerland, according to the Swiss UHPC design code SIA 2052 (École Polytechnique Fédérale de Lausanne 2016).

Material Consistency

Recommendations	Commentary
The fresh UHPC for bridge deck overlays should be thixotropic, so it can be placed on a grade without top forming.	The UHPC supplier will typically adjust the consistency of the UHPC in accordance with the grade. It is recommended to require a demonstration of the UHPC placement on a similar slope to ensure the UHPC material supplier and the installer will not incur any issues on the bridge.

Fiber Content

Recommendations	Commentary
Fiber content should be based on the mechanical properties necessary to meet the strength and serviceability objectives of the bridge deck overlay for preservation or repair.	Most UHPC overlays constructed to date have used 3.25 percent by volume of steel microfibers. This content is based on common practices in Switzerland.

Thickness, Clear Spacing, and Cover

Thickness is the primary design consideration for UHPC overlays. Thickness depends on the performance objectives of the overlay—waterproofing, strengthening, or both. The primary factors influencing the minimum possible thickness of the overlay are the length of the fiber reinforcement and clear cover requirements. The cover and spacing requirements for UHPC overlays differ from those found in the *General Recommendations* subsection, given the installation conditions and rheology of UHPC overlays.

Recommendations	Commentary
The minimum finished overlay thickness should be the greater of 1.0 inch or 1.5 times the maximum fiber length.	Thin lifts of UHPC can results in restricted flow placement and can impede the uniform distribution of fiber reinforcement throughout the overlay. For design, however, a larger minimum value is recommended to allow for construction tolerances and surface profiling. UHPC overlay surfaces are commonly profiled using grinding and grooving for skid resistance.
The minimum nominal clear cover, after finishing and profiling, over reinforcing bars should be 0.625 inch.	This recommendation is based on the practices in Switzerland, as described in SIA 2052 (École Polytechnique Fédérale de Lausanne 2016).
Minimum clear distance between reinforcing bars and the existing concrete deck substrate should be the greater of 0.5 inch or the	Experience has shown that 0.5 inch is sufficient space to ensure consolidation around deck reinforcement when placing and

Recommendations	Commentary
maximum fiber length.	consolidating thixotropic UHPC overlay to reach below the top mat reinforcement (École Polytechnique Fédérale de Lausanne 2016). The engineer should account for the surface roughness profile of the concrete substrate.

Existing Deck Concrete Substrate Preparation

The bond between a UHPC overlay and the existing deck concrete substrate plays a critical role in the performance of the overlaid system. Unlike other applications, composite action between UHPC and concrete is highly desirable. That action relies on the chemical and mechanical bond between the UHPC and concrete. Previous research has demonstrated that this bond is achievable if the substrate concrete has a roughened, prepared amplitude of 0.125 inch minimum (Aaleti and Sritharan 2019). Furthermore, deteriorated cover concrete is also commonly removed in overlay applications. It is critical that concrete roughening or removal methods do not result in microcracking or "bruising" of the prepared substrate, which will reduce the bond strength between UHPC and concrete (Bissonnette et al. 2006). Surface preparation methods that have a lower risk of causing microcracking hydrodemolition are sand blasting and shot blasting (ACI Committee 364 2021), with hydrodemolition and shot blasting being the methods most likely to provide the minimum recommended surface texture. Figure 31 illustrates a substrate surface prepared using hydrodemolition.

Source: FHWA.

Figure 31. Photo. Deck surface prepared by hydrodemolition.

When significant amounts of concrete need to be removed, such as for a partial-depth deck replacement, two removal methods may be combined. For example, cold milling, which tends to have simpler debris management, can be used for initial removal. Final removal and roughening can be completed with hydrodemolition or shot blasting to ensure that no microfractures remain in the substrate. This approach could be particularly advantageous when hydrodemolition is used to limit the amount of wastewater that needs to be treated and disposed.

Recommendations	Commentary
Concrete substrate should have a roughened surface with a minimum profile of 0.125 inch measured as the average distance between the peaks and valleys.	Studies have shown that a minimum 0.125-inch profile is necessary to ensure full composite action (Aaleti and Sritharan 2017). The 0.125-inch profile can be considered equivalent to a roughness average (Ra) of 0.0625, as defined by ASME B46.1 (ASME 2019).

Skid Resistance

The skid resistance of the finished UHPC overlay surface is an important consideration. Hardened UHPC without any texture may be very smooth and may not provide enough friction. As discussed in the *Construction Considerations* subsection, grinding the entire surface of the UHPC is typically used to provide skid resistance. Grinding also eliminates a possible soft top layer resulting from curing with plastic sheeting, eliminates shallow plastic surface cracking that may appear during curing, and provides a smooth riding surface that is not always achieved by the screed.

Recommendations	Commentary
The completed UHPC surface must provide adequate skid resistance.	Skid resistance is typically provided by grinding the entire UHPC surface after curing. Providing skid resistance without grinding may be possible by texturing the surface, but the outcome must be validated based on the proposed texture. ASTM E303 is one possible method to test for skid resistance (ASTM 2018).

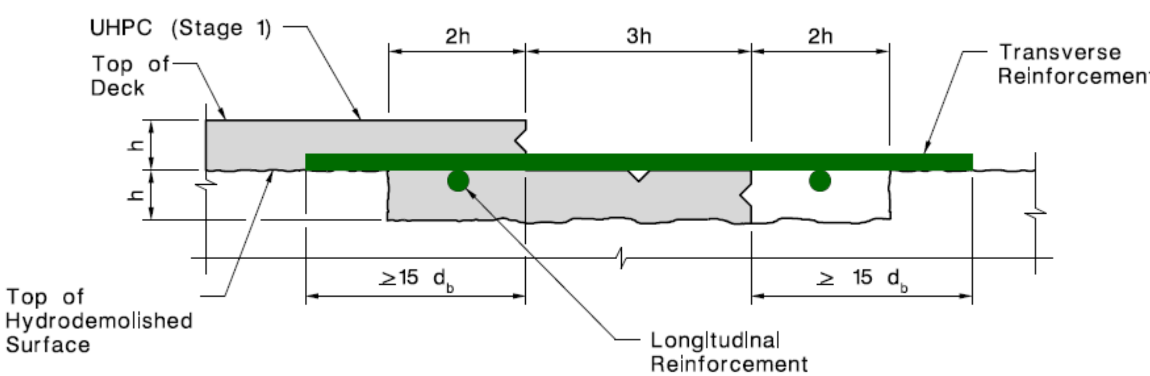
Phased Construction Joints

UHPC overlays are often installed using phased construction in the form of single- or multilane-width stripes. This overlay is commonly installed to maintain traffic on a portion of the bridge or is due to construction equipment limitations. This construction practice results in longitudinal construction joints in between construction phases. Furthermore, long distances between expansion joints may necessitate transverse construction joints in some cases. It is critical in these cases to design and detail phased construction joints to be waterproof but also transfer stress. This step requires proper surface preparation of hardened UHPC to promote bonding with fresh UHPC and potentially the addition of reinforcing bar dowels between phases.

One approach to developing good bond between hardened UHPC and fresh UHPC is to remove the hardened surface cement paste from the hardened UHPC and expose the fiber reinforcement, as shown in figure 32. This result can be accomplished by using an in-form set retarder and pressure washing after the initial set or sandblasting. Research by Jung et al. (2014) demonstrated that this approach can reduce crack width at UHPC-UHPC interfaces.

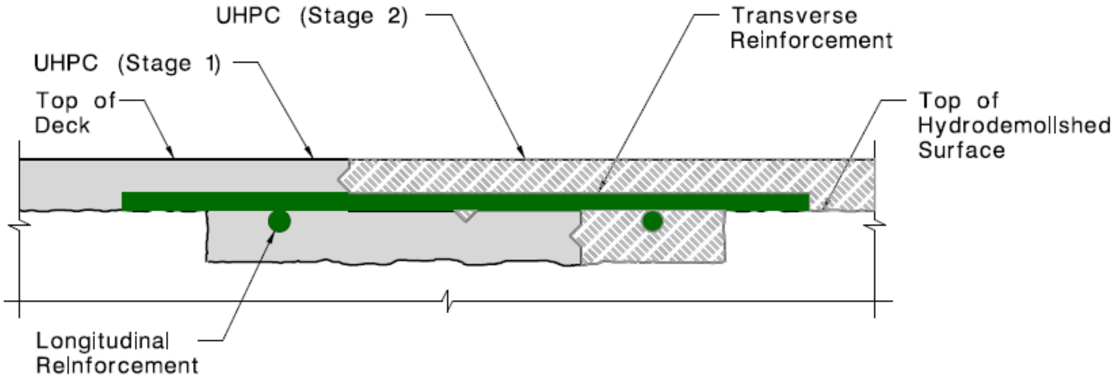
Figure 32. Photo. Exposed fiber finish on a UHPC overlay construction joint.

Reinforcing bars can be placed across the construction joint as both an additional measure to improve the water tightness and a mechanism to transfer forces between phases. This action is particularly critical in areas of negative bending. Consideration should be given to using corrosion-resistant reinforcing bars. Additionally, a stepped construction joint has also been used to enhance water tightness and mechanical interlock between phases. Figure 33 illustrates a common stepped construction joint detail used in Switzerland, which is detailed in their UHPC design guidelines, SIA 2052 (École Polytechnique Fédérale de Lausanne 2016). The detail relies on reinforcing bars to transfer tensile stress across the joint while helping to maintain water tightness when the deck and overlay are loaded by traffic. Additionally, the relatively wide step provides additional area surface for the UHPC-UHPC bond, as well as a longer pathway to reduce the likelihood of any water that might enter the top of the joint reaching the bottom of the joint. This detail was validated during a follow-up investigation of the performance of the UHPC overlay on the bridge over the Morge River (Denarié 2015).



Source: FHWA.

A. Construction of phase 1.

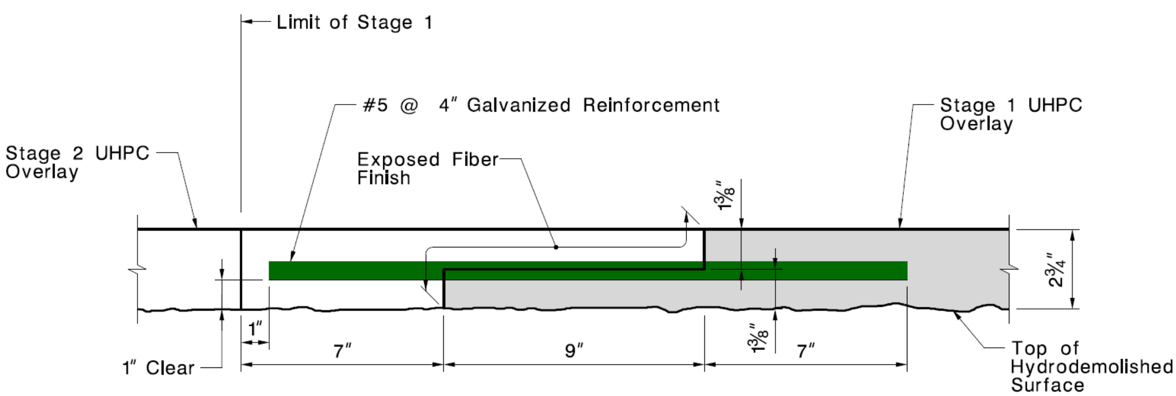


Source: FHWA.

B. Construction of phase 2.

Figure 33. Illustrations. UHPC overlay construction joint detail used in Switzerland (SIA 2052).

A modified version of the Swiss detail has been implemented in the United States on UHPC overlay projects in Iowa and New Jersey. In both cases, modifications eliminated the shear keylike notches and incorporated the step and the reinforcement on top of the roughened deck surface. An example is shown in figure 34. In New Jersey, interface surfaces of the first phase were prepared using an exposed fiber finish before the second phase was placed. One contractor used set retarder on the construction joint formwork with power washing to remove the skin and expose the fiber. Another contractor sandblasted the interface after the UHPC had cured and the edge form was removed, as seen in figure 35.



Source: FHWA.

Figure 34. Illustration. UHPC overlay construction joint on NJ 159 WB Bridge over Passaic River.

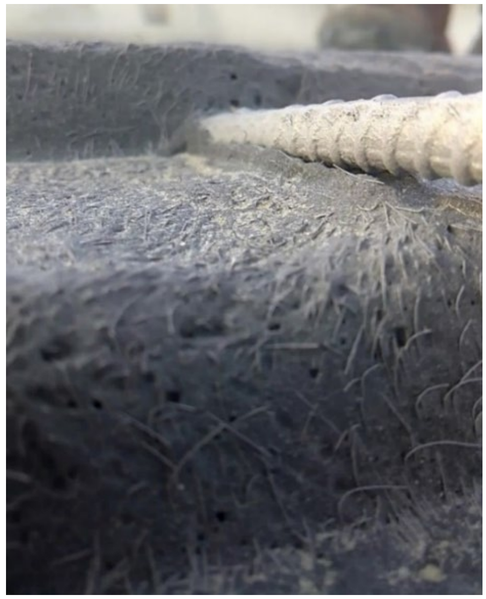


Figure 35. Photo. Stepped UHPC overlay construction joint with exposed fiber finish created by sand blasting on the I–295 Bridge over Mantua Creek in Paulsboro, NJ.

Recommendations	Commentary
Construction joints should be detailed to maximize bond between hardened and fresh UHPC, minimize water intrusion, and provide mechanical continuity.	Several UHPC overlays constructed to date in the United States did not specify construction joint details. At the time of this writing, no performance issues have been reported by the bridge owners. Ideally, the fibers should be exposed on a hardened UHPC surface to enhance the bond between construction phases. Reinforcing bars across the joint may provide further assurance against water infiltration.
Reinforcement of construction joints is recommended if joints are placed in regions subjected to negative bending. The reinforcement provided should be capable of resisting $f_{t,loc}$, before yielding.	Construction joints without reinforcement are more suspectable to premature cracking when placed in regions of negative bending. Furthermore, if a UHPC overlay is being used to strengthen a deck, adding reinforcement across the joint can help ensure the UHPC is able to transfer stress across the construction joint and help ensure that the joint does not become a weak point in the overlay. Reinforcement of construction joints may not be needed if the existing bridge deck reinforcement is fully encapsulated in the UHPC.

Recommendations	Commentary
	Additional guidance on detailing reinforced construction joints is given in the Swiss SIA 2052 guidelines (École Polytechnique Fédérale de Lausanne 2016).

Construction Considerations

Existing Deck Surface Preparation

When an existing concrete deck surface is being prepared for removal, an inspector should review the condition of the deck with respect to the concrete removal method. If the deck is heavily patched, a uniform removal depth may be difficult to achieve with methods such as hydrodemolition and shot blasting due to different strengths of the various patch materials versus the strength of the original deck concrete. For deeper removals, performing initial removals with scarifying is a way to minimize this problem, leaving less patch material for the final hydrodemolition or shot blasting. Hand chipping harder patch areas should be minimized due to the tendency to leave microcracks behind, but, when necessary, the hammer size should be limited to 35 lb maximum. A hydrodemolition or sand blasting hand wand or lance could be used to target isolated high spots without causing microfractures. However, experience has shown that the lance is difficult to control, especially when the blast hits rebar, leading to unintended removals or uncontrolled flying debris.

Testing of Fresh Properties

Currently, no standardized test method exists for testing the fresh properties of thixotropic UHPC for bridge deck overlays and rehabilitation. However, a practical method of testing fresh thixotropic UHPC is to modify the ASTM C1856 flow test by including drops of the flow table (ASTM 2017a). The energy of the drops should cause the UHPC to spread, revealing if the UHPC flows too much or not enough. The number of drops and the acceptable spread should be determined by the UHPC supplier. One supplier is currently using an acceptable spread of 6 to 8 inches after 20 drops of the flow table for placing materials that can be used on slopes up to 6 percent.

Placement Equipment and Methods

The selection of equipment and methods for installing a UHPC overlay are critical to project success. Conventional concrete deck screeds, like that shown in figure 36-A, are used to spread, consolidate, and finish UHPC overlays in cases where the lift thickness is less than 2 inches. The lightweight nature and relatively low power motors that vibrate the screed bar and propel the screed forward are not capable of pushing or driving thick lifts of thixotropic UHPC that have amassed in front of the bar. As such, these screeds require significant assistance from workers with hand tools to spread UHPC to an appropriate thickness before a pass is made with the screed bar. Conventional concrete bridge deck finishing machines are not well-suited for use with thixotropic UHPC overlays. Common issues include UHPC sticking to the augers and rollers and UHPC surface tearing. However, automated bridge deck finishing machines exist that are designed specifically for use with UHPC, as shown in figure 36-B. Automated UHPC paving machines have been used for numerous UHPC overlay installations in the United States.

Source: FHWA.

A. Portable screed bar.

Source: FHWA.

B. Automated UHPC paving machine.

Figure 36. Photos. Common placement equipment for UHPC overlays.

Curing and Finishing

Once the UHPC overlay has been placed, preventing moisture loss and surface dehydration is critical. This outcome is often accomplished by a combination of conventional concrete curing compound and plastic sheeting.

Postconstruction Concerns

The primary postconstruction concern related to UHPC overlays is steel fibers that have been exposed by the grinding and grooving process; this result is illustrated in figure 37-A. There is concern about the fibers being a hazard to vehicular or bicycle tires, as well as being a hazard to pedestrians, pets, or other animals that might walk on the deck surface. At the time of this writing, no occurrences of damage to vehicle or injury to persons or animals have been reported by the bridge owner. Furthermore, UHPC overlays expected to have a grind and groove are commonly not recommended for installation on bridges that might have frequent pedestrian users.

The appearance of the steel fibers on the surface could be a concern, as they corrode due to atmospheric exposure and deicing salts. Over time fibers will rust, and that rust will give the bridge deck a rust-colored hue as they corrode. However, while it is a matter of opinion, distinguishing the rust-colored hue from general dirt and debris on a bridge deck is generally difficult. Additionally, over time, the exposed fibers will disappear, along with the rust-colored hue. This result will occur more rapidly where deicing salts are used and in more heavily traveled lanes. Figure 37-B shows a UHPC overlay after 2 years of service. As shown, the fibers are no longer visible.

A. Shortly after installation.

B. 2 years after service.

Figure 37. Photos. UHPC overlay surfaces after grinding and grooving.

SPECIFIC RECOMMENDATION: LINK SLABS

This subsection provides recommendations for link slab design using UHPC-class materials that meet the minimum properties listed in table 2. The recommendations provided herein are primarily based on procedures and practices used by NYSDOT (New York State Department of Transportation n.d.). NYSDOT was the first agency to develop design, material, and construction specifications for this application. Furthermore, the contents in this subsection will specifically cover UHPC link slabs installed to replace existing expansion joints on in-service bridges, not link slabs installed on new structures. However, many of the same considerations apply in both cases.

Limit States

UHPC link slab design should consider strength, fatigue, and service limit states. The key design parameters include, but are not limited to, link slab thickness, h; debonded length, L_{DB}; and quantity of additional reinforcement, A_s. Detailing these quantities and checking performance for each limit state will require the designer to consider span configuration, bearing type and arrangement, girder end deformations due to live load and other force effects, and bridge skew.

For link slab strength design, the Strength I limit state should be used. In most cases, the primary force effect is live load due to traffic. Dead load of the existing structural element will contribute minimally to the force effect on a link slab due to the construction sequence. However, the designer should also include temperature effects, time-dependent effects such as shrinkage and creep, breaking forces, and the like. An adequate link slab design will likely be controlled by tension localization of UHPC for the strength limit state. That is, when strength load combinations are applied, the design should expect large tensile strains to occur in UHPC.

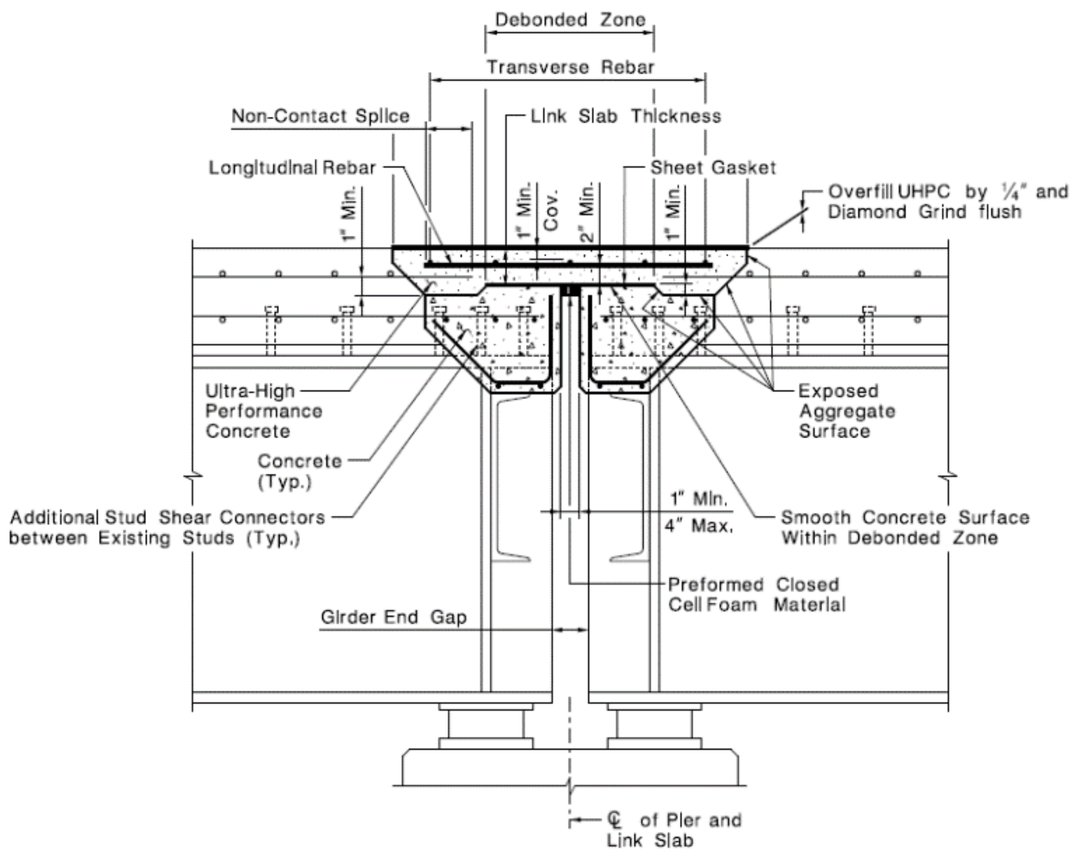
For fatigue design, the designer should primarily consider the fatigue of reinforcing steel placed within the link slab and evaluate the fatigue resistance of reinforcement according to article 5.5.3.1 of the *AASHTO LRFD* (load-and-resistance factor design) using the Fatigue I limit state (AASHTO Highway Subcommittee on Bridges and Structures 2020). Fatigue of UHPC in tension is a potential consideration for the designer. However, it is unlikely to govern or drive the design of the link slab, especially in the case of a link slab that contains steel reinforcement. Furthermore, fatigue of UHPC in tension is still a topic being explored by researchers. The French standard for UHPC design, however, does contain some guidance about UHPC tension fatigue, which may not be appropriate for UHPC link slabs (Association Française de Normalisation 2016).

For service design, the UHPC link slab is expected to crack in tension. The designer should check the stresses and strains in constituent materials under the Service I limit state to determine if these quantities are within an acceptable range. Additional guidance will be given in the following subsections.

Basis of Design and Considerations

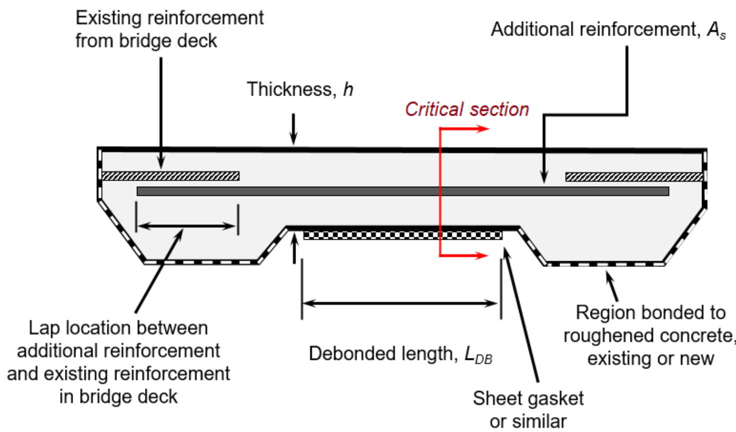
Superstructure movement or deformation in simply supported bridges with conventional expansion joints is accommodated by rotations at the bearing level and translation at the deck level. When a conventional expansion joint is replaced with a link slab, the distribution of superstructure deformation is modified. The expectation is that deformation at the beam ends is accommodated by translation at the bearing level and rotation at the deck level, which occurs within the link slab. As such, the link slab is assumed to be primarily subjected to pure bending. There may be axial effects due to temperature effects or link slab shrinkage. The designer should consider both the local and global behavior of the bridge, including link slabs. Here, “local behavior” refer to the behaviors of the link slab as an individual element, whereas “global behavior” refer to the behaviors of the bridge as a system with the inclusion of the link slabs. One of the primary benefits of a link slab is that simple span behavior, for the most part, can be retained, and girders can function as originally designed. However, replacing conventional expansions with link slabs creates connectivity across the superstructure, which the designer

should consider. For example, long bridges with many simple spans may require relief joints at intermediate supports to accommodate thermal movements that cannot be accommodated by expansion joints at the abutments alone. A structural analysis is required to quantify horizontal or transverse forces acting on existing substructure and foundation elements. The analysis should include any redistribution of braking, wind and seismic loads, as well as potential changes in thermal restraint. The designer should use the results of this analysis to determine if any of the existing substructures, including the foundations, need to be strengthened or replaced.
Figure 38-A depicts typical details for a UHPC link slab relative to other superstructure and substructure elements. Figure 38-B depicts some of the critical design and detailing aspects of a UHPC link slab. The follow subsections provide design recommends that pertain to each.



Source: FHWA.

A. Link slab relative to other superstructure details.



Source: FHWA.

B. Specific details related to the link slab.

Figure 38. Illustrations. Typical link slab details.

Simplified Analysis Procedure

The simplified analysis procedure described herein is similar to that employed by NYSDOT. Engineering judgment should be used to determine the level refinement required for design calculations.

Figure 39 illustrates the key aspects related to the design of the UHPC link slabs. The simplified analysis procedure is as follows:

1. The deformation demand on the link slab is assumed to be the cumulative girder end rotation, θ, caused primarily by vehicle live load. Here, simply supported behavior of the spans is assumed. Note that θ should be based on factored loads that correspond to the limit state of interest.

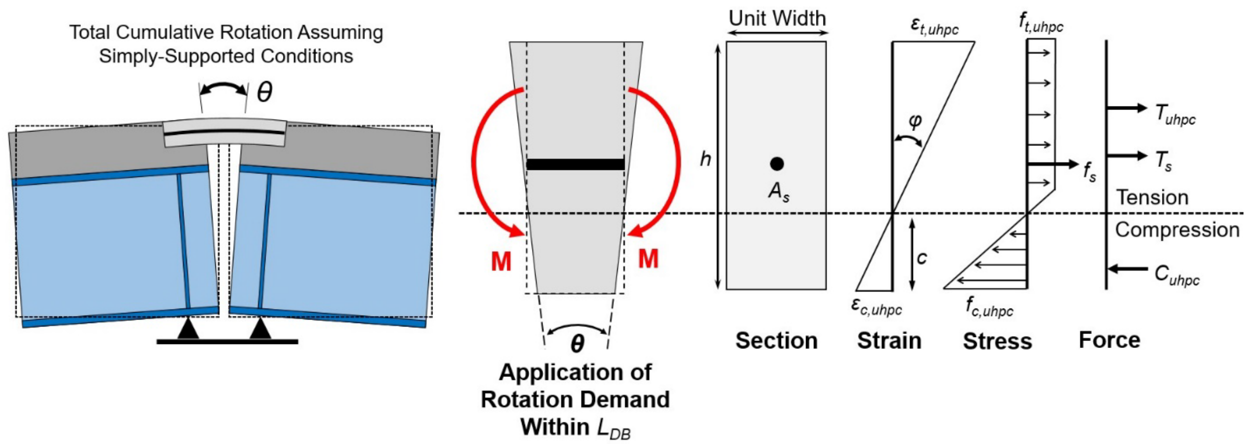
2. The rotation demand is applied to the critical section of the link slab, which is located with the debonded length. After this step, section analysis can be completed using unit width analysis.

3. Assuming that the θ occurs uniformly over the debonded length, the strain distribution in the critical section is assumed to be linear and is defined by a curvature φ, which is equal to L_{DB}/θ, and by assuming a neutral axis location, c.

4. The strain distribution is used to determine the distribution of stresses. The relations between stress and strain for UHPC in compression and tension were presented in the *Stress-Strain Relationships for Design* subsection.

5. The forces in the section are determined based on the stresses in the section, and equilibrium can be checked. The location of c can be iterated until equilibrium of the section is satisfied.

6. The designer should then complete checks associated with the limit state of interest.



Source: FHWA.

M = imposed moment due to θ; $\varepsilon_{t,uhpc}$ = maximum tension strain in UHPC; $\varepsilon_{c,uhpc}$ = maximum compressive strain in UHPC; $f_{c,uhpc}$ = maximum compressive stress in UHPC; $f_{t,uhpc}$ = maximum tension stress in UHPC; f_s = stress in the reinforcing bars; T_{uhpc} = resultant tension force in UHPC, T_s = force in reinforcing bars, C_{uhpc} = resultant compression force in UHPC.

Figure 39. Illustration. Assumed rotation demand and the associated stress, strain, and force distributions in the link slab.

Bridge Geometry and Bearings

Recommendations	Commentary
The maximum skew angle of the bent caps or pier over which a UHPC link slab will be installed should be no greater than 45 degrees.	Large skew angles can impact the local distribution of forces at the end of a bridge deck. This recommendation comes directly from best practices employed in the State of New York.
Bearings should be designed to accommodate deformations occurring at the beam end locations. At least one line of expansion bearings should be used at each link slab location to accommodate girder end movements and rotations. Fixed bearings on each side of the link slab should not be used.	Elastomeric bearings are preferred due to their ability to withstand the repetitive horizontal movements from girder translation and rotation.

Performance at the Service Limit State

Recommendations	Commentary
Except for UHPC in tension, all materials should remain linear-elastic at the service limit state. The tensile strain in UHPC, $\varepsilon_{t,uhpc}$, should be limited to the lesser of $0.25\varepsilon_{t,loc}$ and 0.001.	The UHPC within a debonded zone is expected to undergo tensile strains well beyond the effective first cracking strain. Cracking during service should be limited, and crack widths should be fine. As such,

Recommendations	Commentary
	limiting the tension strain demand on the UHPC to control cracking is needed.

Performance at the Ultimate Limit State

Recommendations	Commentary
The maximum tensile strain in UHPC, $\varepsilon_{t,uhpc}$, should be limited to the localization strain of UHPC, $\varepsilon_{t,loc}$.	At the ultimate limit, the design of the link slab is controlled by localization of UHPC in tension. As such, the tension strain in UHPC at the ultimate limit state should be limited to the localization strain.
All other materials in the link slab should remain linear-elastic at the ultimate limit state.	

Detailing the Debonded Zone

Recommendations	Commentary
A bond breaker should be installed at the interface between the UHPC link slab and any adjacent component within the debonded zone. The length of the debonded zone, L_{DB}, should be designed to accommodate superstructure end rotation.	The debonded zone is the region of the link slab that accommodates superstructure performance and beam end rotation without introducing a significant strain demand in the conventional concrete deck. To achieve this result, a decoupling mechanism must be provided between the link slab and the underlying concrete deck. Commonly a sheet gasket is used.
Reinforcement splicing should not be permitted within the debonded zone.	The debonded zone is expected to undergo tensile strains well beyond the elastic limit of UHPC. As such, tension lap splices or mechanical reinforcement splices are not recommended.
Ancillary elements present at the bridge deck level within the debonded zone, such as barriers, sidewalks, curbs, and the like, should be designed and detailed to accommodate the design θ, without contributing rotational resistance to the link slab.	The debonded zone of a UHPC link slab can be rather short compared with conventional concrete link slabs. As such, ancillary elements, such as barriers, sidewalks, or curbs, that are continuous across the debonded zone will be subject to large, concentrated strains, which are likely to cause excessive cracking of any adjoining concrete components. Decoupling these elements from the debonded zoned will alleviate this demand.

Link Slab Reinforcement and Anchorage

Recommendations	Commentary
Reinforcement located within the debonded zone should have corrosion protection, such as an epoxy coating, or be resistant to corrosion.	Although UHPC-class materials have postcracking ductility, UHPC link slabs are commonly reinforced with steel reinforcing bars. This setup also provides redundancy in the element's ability to withstand tension forces. Furthermore, corrosion protection is highly desirable in the debonded zone, as this region is likely to crack under service loading. This reinforcement is a conservative approach to ensure acceptable performance.
Reinforcement should be anchored outside the debonded zone and should have a lap splice or mechanically spliced continuity with the reinforcement located within the concrete deck. Lap splice should not be permitted in the debonded zone.	This detailing allows the link slab to tie into the reinforcement in the deck to transfer force between the elements.
The portion of the link slab beyond the debonded length is used for anchorage to the concrete deck slab. The interface between UHPC and conventional concrete should be roughened to facilitate bonding.	This region of the link slab accommodates the lap splicing of the link slab and deck reinforcement.
UHPC link slabs are not recommended on decks in poor condition.	

SPECIFIC RECOMMENDATION: BEAM END REPAIR

This subsection provides recommendations for beam end repair using UHPC-class materials that meet requirements listed in table 1. Certain terminologies used in this subsection are specific to describing this repair method. A "panel" is defined as the finished UHPC block that is cast over a portion of the girder. A panel can cover full height or partial height of the web. When two UHPC panels are separated by the bearing stiffener on one side of the web, they are considered to be "adjacent panels." Two panels on opposite sides of the web are considered to be "opposing panels."

Limit States

For UHPC beam end repairs, both strength and fatigue limit states should be considered when the quantity of the stud shear connectors is determined. This repair strategy assumes the in-plane shear in girder webs is shed to UHPC panels through a distribution of stud shear connectors at the girder end, so controlling load combinations are probably based on dead and live loads. That is not to say that load combinations considering horizontal forces (e.g., wind, seismic, and braking forces) may not apply, but the participation of a UHPC beam end repair may be small, if at all.

For strength design, the following three scenarios have been envisioned, depending on the needs of the repair (Kruszewski, Wille, and Zaghi 2018):

- Live load only: This scenario presumes the corroded structure in its current state is satisfactory to resist dead loads, and the repair is only needed to support the live load. Relative to the *AASHTO LRFD* load combinations, this portion of the Strength I load combination is only the live load, or 1.75 times the live load with a 33 percent impact factor (AASHTO Highway Subcommittee on Bridges and Structures 2020).

- Entire design load: For this scenario, the beam end repair has been designed to take the entire factored Strength I load combination, or 1.25 times the dead load of the components plus 1.5 times the dead load of wearing surfaces plus 1.75 times the live load with a 33 percent impact factor.

- Restoration of original capacity: For this scenario, the beam end repair is designed to resist the original capacity of the girder end panel, which may be the shear capacity of the end panel or the capacity of the bearing stiffener. This approach is the most conservative because these values could far exceed the factored design loading, as the proportions of the girder may have been dictated by other factors beyond strength design.

For fatigue design, the shear force range at the beam end should be determined using the *AASHTO LRFD* for both Fatigue I and II load combinations (AASHTO Highway Subcommittee on Bridges and Structures 2020). Fatigue life is an operational decision for how long the repair needs to last and is the structure owner's decision based on the particular bridge. That is, is the repair supposed to keep the bridge operational until full replacement, or is the repair meant to be a permanent solution? This decision also influences which of the three strength design load scenarios is chosen.

For analysis of the loads, any of the methods in the *AASHTO LRFD* are acceptable for the longitudinal and transverse determination of all loads for use in the beam end repair design (AASHTO Highway Subcommittee on Bridges and Structures 2020).

Shear Stud Capacity

Traditionally, stud shear connectors in conventional concrete use a two-part strength equation that is governed by the bearing capacity of the concrete on the stud and rupture of the stud itself. However, the higher strength of UHPC suppresses the bearing capacity of the concrete, and strength is merely controlled by rupture of the stud alone. This process has been demonstrated by push-off test specimens (Cao et al. 2017; Hegger, Rauscher, and Goralski 2004; Kim et al. 2013; Kruzewski, Wille, and Zaghi 2018; Xu et. al. 2022).

Eccentricity in the repair should be minimized by detailing the studs to be symmetric about the girder's bearing reaction force. Individual stud shear connector capacity is reduced as the eccentricity increases (Kruszewski, Wille, and Zaghi 2018). The effect of eccentricity should, therefore, be considered in designing the repair.

Limited fatigue testing of stud shear connectors embedded in UHPC has demonstrated that they fall into the same scatter band of stud shear connectors as in normal-weight concrete.

Recommendations	Commentary
The nominal shear resistance of one stud shear connector embedded in UHPC should be taken as: $$Q_n = \phi_{sc}\,\phi_{ecc}\,0.70A_{sc}F_u \quad (2)$$ Where: ϕ_{sc} = resistance factor for shear connector, which should be taken as 1.00. ϕ_{ecc} = eccentricity reduction factor. A_{sc}= cross-sectional area of a stud shear connector (inch^2). F_u = specified minimum tensile strength of a stud shear connector (ksi).	Stud shear connectors in concrete (normal or UHPC) rupture in a combined stress state between pure shear (0.58 F_u) and pure tension (1.00 F_u). The 0.70 factor has been determined from regression of historical testing of stud shear connectors in conventional concretes. Limited testing of headed shear connectors in UHPC indicates the factor is closer to 1.00, although 0.70 is recommended until more test data are published for stud shear connectors in UHPC. Stud shear connectors should conform to the mechanical requirements of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (AASHTO and AWS 2020). The *Bridge Welding Code* allows for both "type A" and "type B" stud shear connectors, with the difference being magnitude of strength. Beam end repairs could use either type. The design F_u should be commensurate with the chosen stud shear connector type.
Individual stud shear connector strength should be reduced by: $$\phi_{ecc} = 1 - \frac{e}{h} \quad (3)$$ Where: e = overall eccentricity of all stud shear connectors relative to the girder bearing reaction force. h = distance between the lowest and highest stud shear connectors in the repair.	The lever rule should be used to sum the A_{sc} and their horizontal distance from the girder reaction force to determine the overall eccentricity, e. ϕ_{ecc} will provide a conservative reduction for e/h ratios up to 0.50. More exact reductions could be calculated using the instantaneous center of rotation method as described in Kruszewski, Wille, and Zaghi (2018).

Recommendations	Commentary
	Eccentricity is not explicitly known until the shear connectors are fully detailed. Therefore, eccentricity may require some iteration of the design. That is, full symmetry is initially assumed, but eccentricity needs to be evaluated after detailing. The strength design also needs to be rechecked once eccentricity is known.
The fatigue life of an individual stud shear connector should be taken as the minimum of equation 4 and equation 5. $$years = \frac{849{,}162}{n(ADTT)_{SL}} \quad (4)$$ $$years = \frac{A}{(\Delta F)^{m} 365 n(ADTT)_{SL}} \quad (5)$$ Where: *n* = cycles per truck passage. $ADTT_{SL}$ = average daily truck traffic in a single lane. $A = 1{,}040 \times 10^{8}$ (ksi^5). ΔF = the shear stress range of all stud shear connectors. Taken at the Fatigue II shear load range divided by the cross-sectional area of all stud shear connectors. *m* = stud shear connector fatigue growth constant taken as 5.	Equation 4 represents the number of years before Fatigue I (infinite life) would begin to control the design over Fatigue II (finite life). This simplified equation is merely derived by equating Fatigue I and Fatigue II together with their respective load factors and solving for the number of years. If equation 4 is the minimum value, then there are enough stud shear connectors that infinite life can be assumed. Equation 5 represents the calculation of years for finite life using the traditional logarithmic stress-life formulation. Stud shear connectors have been shown to have a different fatigue growth constant over other welded details: they have a fatigue growth constant of 5, whereas all other welded details have a growth constant of 3. If equation 5 is the minimum value, then this is the predicted life of the repair. Cycles per truck passage will likely just be 1.0 for applications of beam end repair.

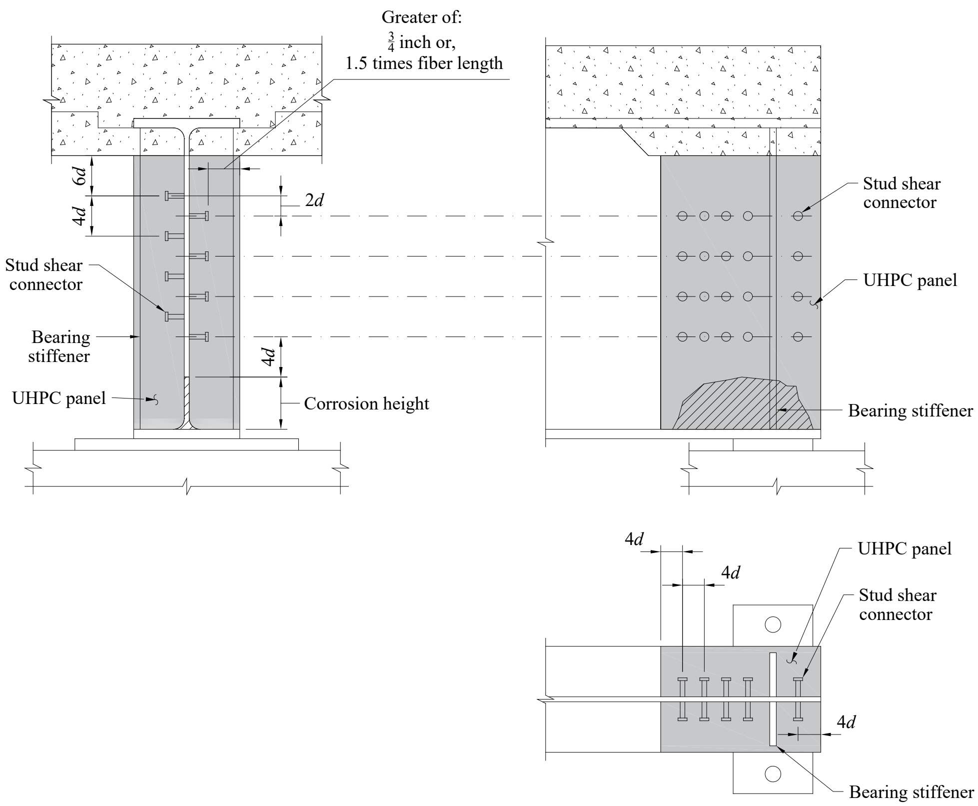
Stud Shear Connector Requirements

Stud shear connectors are typically used to develop composite action between steel girders and concrete decks in flexural shear. In this scenario, the concrete deck and the girder flanges are relatively thick, and the stud shear connectors are welded only to one side of a thick girder flange. In contrast, the beam end repair is a direct shear application of load on the stud shear connectors, which are welded to both sides of a relatively thin girder web. While this process sounds different, detailing stud shear connectors on both side of a girder web is frequently used to anchor the embedded portion of a steel girder in integral abutments. Therefore, the application of stud shear connectors for beam end repairs is not a radical solution. Regardless, stud shear connector size selection and the relative positioning of studs between sides should have some bounds.

Recommendations	Commentary
Each UHPC panel is mechanically attached to the girder.	Stud shear connectors provide mechanical attachment of the UHPC panels to the web. Mechanical attachment may also be achieved using fully threaded rods mechanically fastened through holes in the web or UHPC dowels provided through holes drilled between the web and/or stiffeners to connect opposing or adjacent panels. The diameter of the holes for UHPC dowels is recommended to be at least 1.5 times the fiber length. See Kruszweski and Zaghi (2019) for additional details on alternative mechanical attachments beyond stud shear connectors.
The stud diameter, *d*, should not exceed 2.5 times the thickness of the base metal to which it is attached.	This limit prevents a large-diameter stud from being welded to a thin plate or web.
Studs should only be welded to flat base metal and preferably with no more than 20 percent section loss.	Stud shear connectors welded over heavily pitted steel could suffer from lack of fusion. However, if constraints require welding to pitted steel, the weld soundness should be established by bending the stud shear connector 15 degrees from its original axis after welding. The weld is considered sound if it does not fracture. Section loss reduces the base material thickness. A concern is that distribution of loading between all studs may not be uniform if the based metal thickness is varying. Therefore, not welding to base metal with excessive section loss is preferable. Welding to base metal thinner than 1/8 inch should not be allowed.
A minimum of four stud shear connectors should anchor a panel.	This limit is based on the majority of the push-off specimen data used to derive stud shear connector strength. Detailing requirements were based on specimens with four studs per panel. A good detailing practice would be to use a larger quantity of smaller diameter stud shear connectors to anchor a panel.

Cover and Spacing Requirements

The high strength of UHPC enables the repair design to focus only on the capacity of the stud shear connectors if basic cover requirements are satisfied. Both the full height of the web and the partial height of web repairs are acceptable and capable of restoring lost capacity (Zaghi et al. 2017b). The thickness of the UHPC panel that is cast only needs to cover over the stud shear connectors. The cover limitation is to ensure the development of stud rupture capacity and the flow of UHPC without affecting fiber distribution. Figure 40 shows a schematic for recommended minimum cover and spacing requirements.



Source: FHWA.
Note: Drawing is not to scale.

Figure 40. Illustration. Schematic showing minimum cover and spacing requirements for headed shear connectors in a UHPC beam end repair.

Recommendations	Commentary
Stud shear connectors should have a minimum center-to-center spacing of 4*d* in all directions.[1]	The high strength of UHPC may allow this requirement to be reduced to 3*d*.
Stud shear connectors should have a maximum center-to-center spacing not	This requirement helps to ensure a greater number of smaller diameter studs distributed through the panel in lieu of one stud in each

[1]This requirement will be specified by the next edition of the AASHTO LRFD (10th edition).

Recommendations	Commentary
exceeding 6 inches in all directions.	corner of the panel.
Stud shear connectors should have a minimum side cover of 4*d*.	Laboratory testing demonstrated that a 4*d* cover was needed to develop the rupture capacity of the studs (Kruszewski, Wille, and Zaghi 2018). The UHPC panel exhibited splitting before the studs ruptured in the 2*d* side cover case
Stud shear connectors should have a minimum cover at the top of a partial height panel of 6*d* and 4*d* for full-height panels.	The increase in top cover associated with partial-depth panels is meant to suppress panel splitting failures, considering the potential for air voids at the top of the panel. This problem was not observed once the UHPC fiber content exceeded 2 percent. The cover requirement could be relaxed if needed, provided the UHPC fiber content is equal to or greater than 2 percent. Full-height panels duplicate the side cover requirement for the top cover because some direct load transfer occurs between the UHPC panel and the top flange.
Stud shear connectors should have a minimum spacing of 4*d* from areas of major section loss.	The intent of the repair is to try and weld stud shear connectors to regions with no loss in base metal thickness and thus spaced away from areas of section loss. Other recommendations do allow for stud shear connectors to be welded to regions with limited section loss, although keeping stud shear connectors a distance of *4d* away from base metal with more than 70 percent section loss is advisable.
Stud shear connectors should have a center-to-center spacing of 2*d* between studs on opposite sides of the web.	This provision prevents studs from being welded to the same point on each side of the web so weld heat-affected zones do not overlap. The recommendation is just half of the center-to-center spacing but could be reduced to 1.5*d* if needed. Any further spacing reduction would have to demonstrate that heat-affected zones do not overlap.
Stud shear connectors should have a minimum clear cover the greater of 0.75 inch or 1.5 times the fiber length to the outside face of the UHPC panel.	Sufficient space should be provided for unrestricted flow of UHPC during casting and uniform distribution of fibers.

Bearing Area Requirements

Recommendations	Commentary
The bearing capacity of the UHPC above the girder bearing should be checked to ensure it exceeds the design loading.	Bearing capacity can be checked with *AASHTO LRFD* article 5.6.5. (AASHTO Highway Subcommittee on Bridges and Structures 2020). The effective compressive strength of UHPC, $\alpha_u f'_c$, should be used in equation 5.6.5-2 to determine bearing capacity. The width of the UHPC panel may be greater than that of the girder bearing; therefore, only the projected area of the girder bearing on the UHPC panel should be used for checking the bearing area.

Stud Welding Requirements

The stud shear connector welding should conform to the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (AASHTO and AWS 2020). Base metals other than those listed in the *Bridge Welding Code* should have their weldability established before stud shear connectors are welded to them. Weldability can be assessed by:

- Confirming whether the bridge component already has welded details.
- Checking the carbon equivalent of the base metal via a chemistry test.
- Testing the qualifications of the weldability of the girder itself.

Generally, bridge steel produced after 1974 will be weldable, and steel that is quench and tempered will require special approval because supplemental preheat will be needed to prevent embrittlement of the base metal. Steel with ASTM designations of ASTM A 7, A 8, A 94, and A 440 will certainly need to have weldability established (ASTM 1905, 1912, 1925, 1959). ASTM A 373 steel was developed to be weldable (ASTM 1954). Quench and tempered steels used in bridges that need special attention to preheat and qualification were ASTM A 514, A 517, and A 709 grades 100, 100W, and HPS 100W (ASTM 1964a, 1964b, 2010).

Recommendations	Commentary
Stud shear connectors should be welded according to the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* (AASHTO and AWS 2020).	All provisions specified in clause 9 of AASHTO/AWS D1.5M/D1.5 should be followed, including (AASHTO and AWS 2020): • Material requirements. • Surface preparation. • Welding method. • Weld qualification. • Weld inspection. • Fillet weld repair if 360 degree flash is not achieved.

Recommendations	Commentary
	Welding stud shear connectors with automatically timed stud welding equipment is not prequalified when welding is in the horizontal position. Fillet welding stud shear connectors is allowable, but they are then subject to the requirements of all other clauses in AASHTO/AWS D1.5M/D1.5 (AASHTO and AWS 2020).

Corrosion Mitigation and Inspection

Recommendations	Commentary
Formwork should be detailed such that the UHPC panel completely covers the bottom flange width.	Depending on the girder bottom flange width and the needed stud height, the UHPC panel could be detailed not to cover the entire width of the girder bottom flange. This act could lead to a small portion of the girder bottom flange that could pond water and lead to further corrosion on the line between the UHPC panel and the girder bottom flange. Therefore, completely covering the flange eliminates this risk.
	For girders that are sloped such that water will flow toward the UHPC panel on the girder bottom flange, the leading edge of the UHPC panel in the direction of water flow should be angled such that it sheds water off the bottom girder flange, so water will not be trapped.
UHPC panels should be cast on both sides of the web.	While the designer may elect to utilize a repair that only occupies one side of the web, this practice does not eliminate the risk of corrosion. The bare side of the girder is still susceptible to corrosion. Web section loss below the applied studs may reduce the expected capacity of those studs. Corrosion prevention is maximized when a full-height panel covering the full width of the bottom flange is used.

Recommendations	Commentary
The tops of partial-depth panels should slope away from the web. The edges of the UHPC panel should be caulked.	If separation occurred between the UHPC panel and the steel, ingress of water could result, causing further corrosion behind the UHPC panel. Therefore, sloping the tops of partial-depth repairs away from the web to shed water away is recommended. All edges between UHPC and steel should be caulked.

CHAPTER 5. SPECIFYING UHPC

Like other construction materials, using UHPC requires development of material and construction specifications. Material specifications commonly define the constituents, properties, testing criteria and target performance levels, and testing frequency. The construction specifications provide recommendations for field-related activities such as material storage, formwork adequacy, mock-up requirements, field testing, mixing, placement, and curing. Common to both the material and construction specifications is the need to identify project-specific criteria that may necessitate alterations to the UHPC formulation, mixing process, placing process, or curing process. Lastly, a basis of measurement and payment must be developed that is appropriate for the project.

MATERIALS

At the time of this writing, for a public-sector agency, the most common UHPC material specifications are performance based or product based. Regardless of the specification type, the material specification should identify material tests that must be completed in the field and laboratory to ensure the UHPC product meets the project-specific requirements. To ensure competitive bidding, the Code of Federal Regulations (2011) has requirements for using proprietary products on Federal-aid projects. Using a performance specification and providing competitive bidding of patented items with equally suitable unpatented items are examples of practices that are consistent with these requirements. The regulations also provide focusing proprietary products as experimental features in Federal-aid projects. Additional information on these requirements can be obtained from FHWA division offices.

Performance-Based Specifications

Performance-based specifications are preferred as they encourage innovation and avoid the risk of excluding potential new suppliers or qualified nonproprietary mixes to compete in the marketplace. Performance specifications typically describe a series of material tests that must be completed and an associated set of performance metrics or levels. In some cases, a performance specification may also include a set of mixture proportion ranges that a UHPC product must meet for a chosen application. The specifying agency must determine all the necessary performance criteria, testing methods, and acceptance criteria. In addition, the agency should be confident that the appropriate UHPC constituents and time needed to perform the tests will be available such that the overall schedule of the project will not be affected. Some agencies self-perform or hire independent material testing firms to perform parts or all of the required material testing.

In a performance-based specification, specifying a strict set of mixture proportions would not be appropriate. In addition, this specification is not recommended because most UHPC products available in the United States at the time of this writing are proprietary. As such, the constituent details of those mixes may not be known to the proposing entity. However, the definition of proportion ranges may be helpful to prevent inappropriate products from being proposed to the agency. Examples include limits on water-to-cementitious materials ratio; minimum cement contents; and steel fiber geometry, strength, and percentage by volume supplied in the UHPC mixture.

Product-Based Specifications

Product-based specifications identify a list of preapproved, acceptable products by name. Products are usually proprietary but do not have to be. They are submitted to the agency by suppliers for a particular application. These products perform according to published literature that is based on previously conducted test results. In many jurisdictions, a proprietary-based product can be used only if it undergoes the agency's acceptance-testing protocols and is placed on a preapproved material/product list. Sometimes, an "approved equal" provision is included in the contract plans to allow unidentified suppliers an opportunity to demonstrate whether their product meets the criteria of the contract plans and specifications. Suppliers are typically responsible for maintaining certifications as prescribed by the agency's procurement criteria. Because the United States has multiple UHPC suppliers, a material product specification can be written with competitive bidding on the material as required by many public agencies. Lastly, a product-based specification may have prescriptive language as it relates to material constituents, such as fibers.

Material Testing

It is recommended that material performance tests and acceptance criteria be explicitly identified within the specifications. These criteria provide a means to assess the properties of a particular UHPC mix design and can validate the appropriateness of material characteristics to the intended needs of a project. Manufacturer mix design certification based on prior laboratory testing is appropriate, while material acceptance tests are performed during construction.

ASTM C1856/C1856M, *Standard Practice for Fabricating and Testing Specimens of Ultra-High Performance Concrete*, is the recommended reference for testing most material properties associated with UHPC (ASTM 2017a). Many of the UHPC material property tests described in C1856 are well-established tests used for conventional concrete and include modifications or exceptions needed to apply these tests to UHPC. Thus, ASTM C1856/C1856M references other existing ASTM test procedures and then simply adds the required procedural modifications for UHPC.

The following tests are common for assessing material performance and/or quality assurance (QA/QC) in the field and are included in ASTM C1856/C1856M, unless noted otherwise (ASTM 2017a). Table 3 provides a summary and describes commonly recommended material tests, test frequency, acceptance criteria, and the stage of project delivery when tests are often conducted.

Table 3. Material tests commonly used for vetting and QA/QC of UHPC.

Property	Test Method	Material Vetting	QA/QC in the Field	Field Testing Frequency	Acceptance Criteria
Static flow	ASTM C1856 (C1437 modified) (ASTM 2015b, 2017a)	Yes	Yes	Once per batch	Flow range from 6 to 10 inches
Dynamic flow for UHPC overlays	ASTM C1437 (ASTM 2015b)	Yes	Yes	Once per batch	Project dependent [a]
Compressive strength	ASTM C1856 (C39 modified) (ASTM 2017a, 2020a)	Yes	Yes	At least once per 25 yd^3 or once per 12-hour shift	≥14 ksi before application of construction or live loads [b] ≥18 ksi after 28 days
Direct tension	AASHTO T397 (AASHTO 2022)	Yes	No	N/A	$f_{t,cr} \geq 0.75$ ksi $f_{t,loc} \geq f_{t,cr}$ $\varepsilon_{t,loc} \geq 0.0025$
Freeze-thaw resistance	ASTM C1856 (C666 modified) (ASTM 2015a, 2017a)	Yes	No	N/A	RDM ≥ 90 percent after 300 cycles
Transport properties	ASTM C1856 (C1202 modified) (ASTM 2017a, 2019)[c]	Yes	No	N/A	≤500 Coulombs by 28 days
	AASHTO TP 119 (AASHTO 2021) [c]	Yes	No	N/A	≥1,500 Ω·m by 28 days

[a] Dynamic flow is commonly used to evaluate the rheological properties of thixotropic UHPCs for bridge deck overlays. The acceptable range of dynamic flow depends on the bridge deck geometry, grade, and cross-slope. Based on these factors, the engineer of record should work with the material supplier to establish an acceptable range of flow.
[b] 14 ksi is the strength at which UHPC is mature enough to be fully loaded and at which the rebar development length equations are applicable. In certain situations, this value has been reduced to accommodate certain construction activities and traffic loads, such as overlay applications.
[c] Fiber should be excluded from the test samples.
N/A = not applicable; RDM = relative dynamic modulus of elasticity.

- Flow table testing: The flow table test specified in ASTM C1856 modifies ASTM C1437 and is used to determine the appropriate rheological properties for UHPC (ASTM 2015b, 2017a). Rheological properties describe the flowability or fluidity of a given UHPC mix design. The static spread is commonly used to evaluate highly flowable, self-leveling UHPCs and does not involve table drops. At the conclusion of this test, UHPC can also be visually assessed for fiber distribution and matrix segregation. Thixotropic UHPCs commonly used for bridge deck overlays can also be evaluated using flow table testing but require table drops to assess the flowability under mechanical agitation. ASTM C1856 does not have a standardized procedure for evaluating thixotropic UHPCs. See the subsection *Testing of Fresh Properties* under *Construction Consideration* for more information. Flow table testing is commonly conducted in the laboratory to establish the acceptable spreads for a given project application. It is also commonly conducted in the field immediately after mixing to assess the mix before placement, to address environmental changes that may influence the mix, and to ensure consistency among batches.

- Compressive strength: Compression testing is specified in ASTM C1856, modifying ASTM C39, and is used to evaluate the compression response of UHPC (ASTM 2015b, 2017a). Compression testing of UHPC, per ASTM C1856, requires 3-inch-diameter by 6-inch-long cylinders. Reduced-size specimens are used primarily to enable the use of lower capacity compression-testing equipment without any significant variation in results compared with full-size cylinders. Compression testing is conducted at a given age to determine strength. If laboratory testing is used to determine curing behavior of field-cast UHPCs, then the cylinders should be match cured similar to the anticipated field conditions. Cylinders are typically tested for at least two different compression strength criteria. The first criterion is minimum compressive strength for removal of formworks and application of construction loads, and the second criterion is the mature compressive strength, which is commonly measures at 14 or 28 days.

- Direct tension strength: Direction tension testing should be conducted according to the procedure described by AASHTO T397 (AASHTO 2022). This test is used to quantify the tension behavior of UHPC-class materials by determining the effective first cracking stress, stress-hardening behavior, and localization stress and strain. Tension testing is conducted using prismatic specimens that measure 17 inches in length and have cross-sectional dimensions of 2 inches by 2 inches. This testing can be completed in most uniaxial load frames commonly found in academic or commercial testing labs. This test does require the performing lab to be able to capture axial stress and strain over a specified gauge length simultaneously. Direct tension testing is commonly using during the material vetting or prequalification process and is not common for QC or QA testing.

- Freeze-thaw resistance: The freeze-thaw resistance test specified in ASTM C1856 modifies ASTM C666 and is used to make empirical determinations regarding the freeze-thaw durability of a given UHPC (ASTM 2015a, 2017a). ASTM C1856 specifies the resistance to freezing and thawing should be testing using procedure A, except that each specimen shall be subjected to at least 300 cycles or the number needed until its relative dynamic modulus of elasticity, as defined in ASTM C666, reaches 90 percent, whichever occurs first, unless other limits are specified. Freeze-thaw resistance testing is commonly using during the material vetting or prequalification process and is not common for QC or QA testing.

- Transport properties: The transport properties describe how moisture and ions move through the pore network of a cementitious material and can be directly related to a material's long-term durability. Numerous standard test methods are available to assess transport properties of cementitious materials; some are applicable to UHPC-class materials. Historically, the rapid chloride ion penetrability test, as outlined in ASTM C1202/1202M, has been used (ASTM 2019). One test that is gaining traction for describing the transport properties is a resistivity test, described by AASHTO (2021), TP 119 *Standard Method of Test for Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test*. A threshold of 1,500 Ω·m has been identified as being indicative of UHPC-class materials (Spragg and Graybeal 2022). Both of these tests are electrical based. When used to assess the transport properties of a UHPC, they should be conducted using specimens with no steel fibers. Steel fibers, even at low volume fractions, can lower the impact of the measured values in this test and are not indicative of the microstructural quality. Because many of these tests were developed for conventional concretes, modifications to the testing procedure, most commonly to the testing time, are often needed to accurately characterize the dense microstructure typical of UHPCs. Transport property testing is typically used during the material vetting or prequalification process and is not common for QC or QA testing.

CONSTRUCTION

In-field activities related to the storage, mixing, and placement of UHPC are often detailed in the construction portion of UHPC specifications. Some specifications require contractors to demonstrate competence with mixing and placing UHPC materials before the start of actual work using a mock-up. Construction specifications may include trial mixing, specification of the order and manner of constituent mixing, mixing equipment, placement, testing, adequacy of formwork, bond surface preparation, and demonstration of appropriate staffing and equipment. If a proprietary UHPC is chosen for a project, specifications typically require that a representative of the supplier be onsite during UHPC mixing and placing operations. Construction specifications may also prescribe curing methods, onsite material storage, and material-usage time frames (such as expiration dates). The follow items are commonly included in the construction specification:

- Qualifications related to contractor experience and capabilities.
- Requirements related to contractor prequalification by completion of construction mock-ups.
- Determination of whether a prepour meeting is required, who must attend, and when.
- Requirements related to material storage location, humidity, and temperature.
- Specific requirements related to adequacy of surface preparation for bond and formwork.
- Specific requirements related to UHPC mixing, placing, finishing, and curing.
- Frequency of strength gain and material testing/sampling.

INSPECTION

Monitoring construction activities is a critical part of the deployment of field-cast UHPC. As with any phase of a bridge construction project, this phase requires close attention to the process and the generation of appropriate documentation. The construction engineering inspector must be familiar with the overall construction process, as well as with specific differences associated with the deployment of this technology compared with conventional grouts and concretes. Some examples of specific items to consider are as follows:

- Familiarity with specifications for UHPC and any other relevant special provisions.
- Worker safety equipment and procedures.
- Test placement involving batching, mixing, testing, and placing UHPC.
- Lot numbers, dates, and storage of constituent materials.
- Mixing process, including weighing, timing, and discharging.

- Formwork considerations.
- Surface preparation.
- Field test methods for assessing fresh properties.
- UHPC placement.
- Form closure requirements, including over pressure.
- Curing requirements.
- Surface preparation of hardened UHPC.

These specific items have been discussed throughout this subsection. FHWA has published a comprehensive example construction checklist for UHPC connections for prefabricated bridge elements (Graybeal and Leonard 2018). Many of the items in the example checklist will apply to UHPC bridge P&R applications, so owners may want to review this checklist when developing their agency's UHPC inspection procedures.

MEASUREMENT AND PAYMENT

The unit of measurement for the quantity of UHPC used on a project and the payment method associated with the measurement are important factors to consider when developing a specification. The measurement and payment method will associate risk with either the owner or the contractor. Furthermore, the measurement and payment method should be commensurate with the application and should consider factors that may result in financial risk to one party versus another, namely the contractor or the owner. A few examples are listed as follows:

- Payment by volume: The contractor is paid for the volume of UHPC installed. In this method, the contractor assumes less risk and is likely to produce a more accurate bid. In this case, field measurements are critical, and the owner assumes more risk. This methodology could be used for virtually any project.

- Payment by length or surface area: The contractor is paid per linear length or per unit of surface area of UHPC installed. In this method, the contractor assumes more risk and is likely to produce a more conservative (costly) bid. In this case, field measurements of material usage are not as critical, and the owner assumes less risk. This methodology would be most appropriate for projects where the geometry of the UHPC element to be created is well defined, such as link slabs, headers, or overlay.

- Payment by unit: The contractor is paid per UHPC element installed. In this case, the size and number of UHPC elements to be created must be well defined. Here, field measurements of material usage are not as critical. This methodology would be most appropriate for projects where numerous UHPC elements of similar size are to be created, such as steel beam end repairs.

CHAPTER 6. CONCLUDING REMARKS

UHPC provides new opportunities and solutions for bridge P&R. This report provides an overview of different UHPC-based bridge P&R solutions and includes design and construction recommendations for a select set of these technologies that has been proven based on research findings and field pilot projects. These projects include UHPC for bridge deck overlays, UHPC link slabs, and UHPC beam end repairs. This document is expected to facilitate wider deployment of UHPC P&R applications by providing owners, designers, and contractors the information necessary to develop agency specifications and adopt UHPC-based solutions for P&R of bridges.

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REFERENCES

Aaleti, S., B. Petersen, and S. Sritharan. 2013. *Design Guide for Precast UHPC Waffle Deck Panel System, Including Connections*. Report No. FHWA-HIF-13-032. Washington, DC: Federal Highway Administration, p. 125.

Aaleti, S., and S. Sritharan. 2017. *Investigation of a Suitable Shear Friction Interface Between UHPC and Normal Strength Concrete for Bridge Deck Applications*. Project No. 10-379. Ames, IA: Iowa Department of Transportation.

Aaleti, S., and S. Sritharan. 2019. "Quantifying Bonding Characteristics between UHPC and Normal Strength Concrete for Bridge Deck Application." *Journal of Bridge Engineering* 24, no. 6.

AASHTO. 2021. *Standard Method of Test for Electrical Resistivity of a Concrete Cylinder Tested in a Uniaxial Resistance Test*. TP 119-21. Washington, DC: American Association of State Highway and Transportation Officials.

AASHTO. 2022. *Standard Method of Test for Uniaxial Tensile Response of Ultra-High Performance Concrete*. T 397-22. Washington, DC: Washington, DC: American Association of State Highway and Transportation Officials (Forthcoming).

AASHTO and AWS. 2020. *Bridge Welding Code*. AASHTO/AWS D1.5M/D1.5. Miami, FL: American Welding Society.

AASHTO Highway Subcommittee on Bridges and Structures. 2020. *AASHTO LRFD Bridge Design Specifications*, 9th ed. Washington, DC: AASHTO.

ACI. 2014. *Guide to Hot-Weather Concreting (ACI 305R-14)*. Farmington Hills, MI: American Concrete Institute.

ACI Committee 364. 2021. *Evaluation and Minimization of Bruising in Concrete Repair*. Technote No. PRC-364.7-21 Farmington Hills, MI: American Concrete Institute.

ASME. 2019. *Standard B46.1. Surface Texture (Surface Roughness, Waviness, and Lay)*. New York, NY: American Society of Mechanical Engineers.

Association Française de Génie Civil. 2013. *Bétons Fibrés à Ultra-Hautes Performances, Recommandations* [*Ultra High Performance Fiber-Reinforced Concretes, Recommendations*]. Paris, France: AFGC Groupe de travail BFUP. https://www.afgc.asso.fr/app/uploads/2007/10/Recommandations-BFUP-Juin-2013.pdf, last accessed September 8, 2020.

Association Française de Normalisation. 2016. *French National Standard (NF) P18-470. Ultra-High Performance Fiber-Reinforced Concrete—Specifications, Performance, Production and Conformity*. Paris, France: Association Française de Normalisation (AFNOR).

ASTM. 1905. “Standard Specifications for Structural Steel for Bridges, ASTM A 7,” in *Proceedings of the Eighth Annual Meeting for Testing and Materials, Atlantic City, NJ*. Philadelphia, PA: American Society for Testing Materials.

ASTM. 1912. “Standard Specifications for Structural Nickel Steel, ASTM A 8,” in *Proceedings of the Fifteenth Annual Meeting, American Society for Testing and Materials, New York, NY*. Philadelphia, PA: American Society for Testing Materials.

ASTM. 1925. “Tentative Specifications for Structural Silicon Steel, ASTM A 94,” in *Proceedings of the Twenty-Eighth Annual Meeting, American Society for Testing and Materials, Atlantic City, NJ*. Philadelphia, PA: American Society for Testing Materials.

ASTM. 1954. “Tentative Specification for Structural Steel for Welding, ASTM A 373,” in *ASTM Standards–Part I Ferrous Metals*. Philadelphia, PA: American Society for Testing and Materials.

ASTM. 1959. “Tentative Specification for High-Strength Structural Steel, ASTM A 440,” in *ASTM Standards–Part I Ferrous Metals*. Philadelphia, PA: American Society for Testing and Materials.

ASTM. 1964a. “Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding, ASTM A 514,” in *Annual Book of ASTM Standards, Part 4*. Philadelphia, PA: American Society for Testing and Materials.

ASTM. 1964b. “Standard Specification for Pressure Vessel Plates, Alloy Steel, High-Strength, Quenched and Tempered, ASTM A 517,” in *Annual Book of ASTM Standards, Part 4*. Philadelphia, PA: American Society for Testing and Materials.

ASTM. 2010. *Standard Specification for Structural Steel for Bridges*. ASTM A709-10. West Conshohocken, PA: ASTM International.

ASTM. 2013a. *Standard Test Method for Density, Absorption, and Voids in Hardened Concrete*. C642A. West Conshohocken, PA: ASTM International.

ASTM. 2013b. *Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method).* C1567. West Conshohocken, PA: ASTM International.

ASTM. 2014a. *Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression*. C469/C469M. West Conshohocken, PA: ASTM International.

ASTM. 2014b. *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method).* C1260. West Conshohocken, PA: ASTM International.

ASTM. 2015a. *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing.* C666/C666M. West Conshohocken, PA: ASTM International.

ASTM. 2015b. *Standard Test Method for Flow of Hydraulic Cement Mortar.* C1437. West Conshohocken, PA: ASTM International.

ASTM. 2016. *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance.* C403/C403M. West Conshohocken, PA: ASTM International.

ASTM. 2017a. *Standard Practice for Fabricating and Testing Specimens of Ultra-High Performance Concrete*. C1856/C1856M. West Conshohocken, PA: ASTM International.

ASTM. 2017b. *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete*. C157/C157M. West Conshohocken, PA: ASTM International.

ASTM. 2018. *Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester*. E303. West Conshohocken, PA: ASTM International.

ASTM. 2019. *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*. C1202. West Conshohocken, PA: ASTM International.

ASTM. 2020a. *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. C39/C39M. West Conshohocken, PA: ASTM International.

ASTM. 2020b. *Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)*. C1583/C1583M. West Conshohocken, PA: ASTM International.

Berry, M., R. Snidarich, and C. Wood. 2017. *Development of Non-Proprietary Ultra-High Performance Concrete*. Report No. MT-17-010/8237-001. Helena, MT: Montana DOT.

Bissonnette, B., L. Courard, A. Vaysburd, and N. Bélair. 2006. "Concrete Removal Techniques: Influence on Residual Cracking and Bond Strength." *Concrete International* 28, no. 12: 49–55.

Blais, P., and M. Couture. 1999. "Precast, Prestressed Pedestrian Bridge—World's First Reactive Powder Concrete Structure." *PCI Journal* 44, no. 5: 60–71.

Brühwiler, E., and E. Denarié. 2013. "Rehabilitation and Strengthening of Concrete Structures Using Ultra-High Performance Fibre Reinforced Concrete." *Structural Engineering International* 23, no. 4: 450–457.

Brühwiler, E., H. Mühlberg, B. Houriet, B. Fleury, S. Cuennet, P. Schär, F. Boudry, and M. Maurer. 2015. "Strengthening the Chillon Viaducts Deck Slabs with Reinforced UHPFRC," in *Proceedings of IABSE Conference 2015*. New York, NY: Curran Associates, p. 1171–1178.

Caner, A., and P. Zia. 1998. "Behavior and Design of Link Slabs for Jointless Bridge Decks." *PCI Journal* 43, no. 3: 68–80.

Cao, J., X. Shao, L. Deng, and Y. Gan. 2017. "Static and Fatigue Behavior of Short-Headed Studs Embedded in a Thin Ultrahigh-Performance Concrete Layer." *Journal of Bridge Engineering* 22, no. 5: 04017005.

Code of Federal Regulations. 2011. "Material or Product Selection," 23 CFR § 635.411. https://www.govinfo.gov/content/pkg/CFR-2011-title23-vol1/pdf/CFR-2011-title23-vol1-sec635-411.pdf, last accessed October 13, 2018.

Dagenais, M.-A., B. Massicotte, and G. Boucher-Proulx. 2018. "Seismic Retrofitting of Rectangular Bridge Piers with Deficient Lap Splices Using Ultrahigh-Performance Fiber-Reinforced Concrete." *Journal of Bridge Engineering* 23, no. 2: 04017129.

De La Varga, I., Z. Haber, and B. Graybeal. 2017. *Bond of Field-Cast Grouts to Precast Concrete Elements*. Report No. FHWA-HRT-16-081. Washington, DC: Federal Highway Administration, p. 19.

Denarié, E. 2015. *Pont sur la Morge–campagne de caractérisation de l'état de l'ouvrage 9.5 ans après intervention [Bridge over the River Morge–Investigation of the Bridge Condition 9.5 Years After Rehabilitation]*. Report No. MCS 23.14.05-1. Lausanne, Switzerland: Ecole Polytechnique Fédérale de Lausanne.

Doiron, G. 2019. "UHPC Shotcrete—A New Way to Install UHPC." *Second International Interactive Symposium on Ultra-High Performance Concrete* 2, no. 1.

École Polytechnique Fédérale de Lausanne. n.d. "UHPFRC Map Switzerland" (web page). https://www.epfl.ch/labs/mcs/mcs-laboratory-for-maintenance-and-safety-of-structures/uhpfrc-map-switzerland/, last accessed July 7, 2021.

École Polytechnique Fédérale de Lausanne. 2016. *Recommendation: Ultra-High Performance Fibre Reinforced Cement-Based Composites (UHPFRC), Construction Material, Dimensioning and Application, an English translation of the Swiss Society of Engineers and Architects (SIA) Technical Leaflet SIA 2052 with adaptations*. Lausanne, Switzerland: EPFL.

El-Helou, R., Z. Haber, and B. Graybeal. 2022. "Mechanical Behavior and Design Properties of Ultra-High Performance Concrete." *ACI Materials Journal* 119, no. 1, 181–194, 10.14359/51734194.

El-Tawil, S., Y.-S. Tai, B. Meng, W. Hansen, and Z. Liu. 2018. *Commercial Production of Non-Proprietary Ultra High Performance Concrete*. Report No. RC-1670. Ann Arbor, MI: Michigan Department of Transportation.

Esmaeili, J., and J. Kasaei. 2016. "Effect of Different Curing Regimes on Strength and Transport Properties of UHPC Containing Recycled Steel Tire Wires as Micro Steel Fibers." *First International Interactive Symposium on Ultra-High Performance Concrete* 1, no. 1.

FHWA. 2018. *Bridge Preservation Guide: Maintaining a Resilient Infrastructure to Preserve Mobility*. Report No. FHWA-HIF-18-022. Washington, DC: Federal Highway Administration.

FHWA. 2021. "Ultra-High Performance Concrete—Extending the Life of the Nation's Bridges." *Innovator* 14, no. 83. https://www.fhwa.dot.gov/innovation/innovator/issue83/page_01.html, last accessed October 4, 2021.

FHWA. 2022. "Deployments of UHPC in Highway Bridge Construction. Interactive Map" (web page). https://highways.dot.gov/research/structures/ultra-high-performance-concrete/deployments, last accessed February 25, 2022.

Graybeal, B. 2019. *Design and Construction of Field-Cast UHPC Connections*. Report No. FHWA-HRT-19-011. Washington, DC: Federal Highway Administration.

Graybeal, B., and F. Baby. 2013. "Development of Direct Tension Test Method for Ultra-High-Performance Fiber-Reinforced Concrete." *ACI Materials Journal* 110, no. 2: 177–186.

Graybeal, B., and M. Leonard. 2018. *Example Construction Checklist: UHPC Connections for Prefabricated Bridge Elements*. Report No. FHWA-HIF-18-030. Washington, DC: Federal Highway Administration.

Haber, Z. B., I. De la Varga, B. Graybeal, B. Nakashoji, and R. El-Helou. 2018. *Properties and Behavior of UHPC-Class Materials*. Report No. FHWA-HRT-18-036. Washington, DC: Federal Highway Administration.

Haber, Z. B., and B. Graybeal. 2019. "Advancing Bridge Repair and Preservation Using Ultra-High-Performance Concrete." *Aspire* Spring: 45–46.

Haber Z. B., J. Munoz, and B. Graybeal. 2017. *Field Testing of an Ultra-High Performance Concrete Overlay*. Report No. FHWA-HRT-17-096. Washington, DC: Federal Highway Administration.

Haikal, G., J. A. Ramirez, M. R. Jahanshahi, S. Villamizar, and O. Abdelaleim. 2019. *Link Slab Details and Materials*. Joint Transportation Research Program Report No. FHWA/IN/JTRP-2019/10. West Lafayette, IN: Purdue University.

Hain, A., A. E. Zaghi, T. Fields, R. Barakat, A. Cardinali, M. Culmo, and T. Lopata. 2019. "Implementation of UHPC for the Repair of a Steel Bridge with Corrosion Damage in Connecticut, USA." *Second International Interactive Symposium on Ultra-High Performance Concrete* 2, no. 1: 1–11.

Hegger, J., S. Rauscher, and C. Goralski. 2004. "Push-Out Tests on Headed Studs Embedded in UHPC," in *Proceedings of the International Symposium on Ultra High Performance Concrete*, *Kassel, Germany*, vol. 3, p. 425–434.

Jung, K., S. Park, S. Kim, B. Kim, and B. Cho. 2014. *A Study on the Flexural Performance of HPC Precast Deck-Joint Interface by the Exposure of Steel Fiber, Engineering.* Wuhan, China: Scientific Research Publishing Inc., vol. 2014, no. 6, p. 1000–1006.

Kim, J. S., S. H. Park, C. K. Joh, and E. S. Choi. 2013. "Push-Out Test on Shear Connectors Embedded in UHPC." *Applied Mechanics and Materials* 351–352: 50–54.

Kono, K., H. Musha, T. Kawaguchi, A. Eriguchi, S. Tanaka, T. Kobayashi, and M. Ikeda. 2013. "Durability Study of the First PC Bridge Constructed with Ultra High Strength Fiber Reinforced Concrete in Japan," in *Proceedings of the RILEM-fib-AFGC International Symposium on Ultra-High Performance Fibre-Reinforced Concrete*, ed. F. Toutlemonde and J. Resplendino. Begneux, France: RILEM Publications, p. 239–248.

Kruszewski, D. 2018. "Performance Evaluation of Shear Connectors Embedded in Ultra-High Performance Concrete as Part of a Bridge Repair Method." Doctorate dissertation. University of Connecticut. https://opencommons.uconn.edu/dissertations/1963, last accessed November 24, 2021.

Kruzewski, D., K. Wille, and A. Zaghi. 2018. "Push-out behavior of headed shear studs welded on thin plates and embedded in UHPC." *Engineering Structures* 173: 429–441.

Kruszewski, D., and A. Zaghi. 2019. "Design of Various Shear Connectors for Repair of Corroded Steel Girders with Ultra-High Performance Concrete." *Transportation Research Record* 2673, no. 2: 521–530.

Mohebbi, A., B. Graybeal, and Z. Haber. 2022. "Compressive Creep and Unrestrained Shrinkage of Ultra-High Performance Concrete: Time-Dependent Properties Affecting Prestress Losses." *ASCE Journal of Civil Engineering Materials* (Forthcoming).

New York State Department of Transportation. n.d. "Link Slab Design" (web page). https://www.dot.ny.gov/main/business-center/designbuildproject39/repository/Link%20Slab%20Design%20EX.pdf, last accessed March 15, 2020.

Piérard, J., B. Dooms, and N. Cauberg. 2013. "Durability Evaluation of Different Types of UHPC," in *Proceedings of the RILEM-fib-AFGC International Symposium on Ultra-High Performance Fibre-Reinforced Concrete*, ed. F. Toutlemonde and J. Resplendino. Begneux, France: RILEM Publications, p. 275–284.

Sbia, L., A. Peyvandi, I. Harsini, S. Abideen, R. Weerasiri, A. Balachandra, and P. Soroushian. 2017. "Study on Field Thermal Curing of Ultra-High-Performance Concrete Employing Heat of Hydration." *ACI Materials Journal* 114, no. 5: 733–743.

Shafei, B., B. Phares, and W. Shi. 2020. *"Beam End Repair for Prestressed Concrete Beams*." InTrans Project 17-598. Ames, IA: Iowa Department of Transportation.

Spragg, R. P., and B. Graybeal. 2022. *Electrical Resistivity Testing to Rapidly Assess the Durability of UHPC-Class Materials*. Report No. FHWA-HRT-21-095. Washington, DC: Federal Highway Administration.

Sritharan, S. 2015. "Design of UHPC Structural Members: Lessons Learned and ASTM Test Requirements." *Advances in Civil Engineering Materials* 4: 113–131.

Sritharan, S., G. Doiron, D. Bierwagen, B. Keierleber, and A. Abu-Hawash. 2018. "The First Application of UHPC Bridge Deck Overlay in North America." *Transportation Research Record* 2672, no. 26: 40–47.

Summers, J. 2021. " Use of Ultra-High Performance Concrete on the Commodore Barry Bridge" (web page). https://www.roadsbridges.com/use-ultra-high-performance-concrete-commodore-barry-bridge, last accessed October 4, 2021.

Thomas, M., B. Green, E. O'Neal, V. Perry, S. Hayman, and A. Hossack. 2012. "Marine Performance of UHPC at Treat Island," in *Ultra-High Performance Concrete and Nanotechnology in Construction. Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, ed. M. Schmidt, E. M. Fehling, C. Glotzbach, S. Fröhlich, and S. Piotrowski. Kassel, Germany: Kassel University Press, p. 365–370.

Thorkildsen E., and Greenman Pedersen Inc. 2020. *Case Study: Eliminating Bridge Deck Joints with Link Slabs—An Overview of State Practices*. Report No. FHWA-HIF-20-062. Washington, DC: Federal Highway Administration.

Wille, K., and C. Boisvert-Cotulio. 2013. *Development of Non-Proprietary Ultra-High Performance Concrete for Use in the Highway Bridge Sector*. Report No. FHWA-HRT-13-100. Washington, DC: Federal Highway Administration.

Xu, Q., W. Sebastian, K. Lu, Y. Yao, and J. Wang. 2022 "Parametric Experimental Study of Ultra-Short Stud Connections for Lightweight Steel–UHPC Composite Bridges." *Journal of Bridge Engineering* 27, no. 2: 04021108.

Yuan, J., and B. Graybeal. 2014. *Bond Behavior of Reinforcing Steel in Ultra-High Performance Concrete*. Report No. FHWA-HRT-14-089. Washington, DC: Federal Highway Administration.

Zaghi, A. E., K. Wille, K. Zmetra, and K. McMullen. 2017a. *Repair of Steel Beam/Girder Ends with Ultra High-Strength Concrete (Phase I): Final Report*. Report No. CT-2282-F-15-2. Storrs, CT: University of Connecticut.

Zaghi, A. E., K. Wille, K. Zmetra, K. McMullen, D. Kruszewski, and A. Hain. 2017b. *Repair of Steel Beam/Girder Ends with Ultra High-Strength Concrete—Phase II*. Report No. CT-2295-1-17-2. Storrs, CT: University of Connecticut.

Zuk, W. 1982. *Jointless Bridges*. Report No. FHWA/VA-81/48. Charlottesville, VA: Virginia Highway and Transportation Research Council.

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