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EXPLORATION OF UHPC APPLICATIONS FOR MONTANA BRIDGES

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THE U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

February 2024

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16. Abstract

The following research project explores bridge applications of ultra-high performance concrete (UHPC). Bridge deterioration, including decks and other members, is a problem across Montana and UHPC overlays and patching/repairing have been found to be viable alternatives to bridge/element replacement. The current study began with a literature review on research, specifications, and implementation projects of UHPC bridge deck overlays. Existing UHPC overlay projects were reviewed and lessons learned by other states while using and specifying UHPC for bridge deck overlays were investigated. A recently published report from FHWA was highlighted, as it summarized the results of previous overlay and repair projects, and developed recommendations for the successful implementation in these applications. The highlighted content included overall material specifications for UHPC and design and construction specific considerations. A material-level evaluation was performed on three UHPC mixes, primarily focusing on workability, compressive strength, tensile strength, and tension and shear bond strengths. All three UHPCs exhibited adequate behavior and the resultant properties were above recommendations from ACI for concrete repair and overlay applications. Based on the material level evaluation results, a thixotropic version of Ductal was chosen for subsequent structural testing. Five slab test specimens were designed and constructed to model a deck section from an existing bridge in Montana. The testing and specimens were designed to determine the effects that including a UHPC overlay, overlay thickness, and substrate concrete strength have on the ultimate moment capacity. The slabs consisted of one control slab, two slabs with varying UHPC overlay depths, one with weak substrate concrete, and one tested to emulate a negative moment region on a bridge deck (with UHPC overlay subjected to tension during bending). The testing demonstrated that including a UHPC overlay increased the ultimate moment capacity of the slabs, even with a weak substrate concrete. Slabs with a UHPC overlay typically experienced a shear failure at a high level of displacement and well after steel yielding, compared to a flexural failure in the control slab with concrete crushing at midspan. Additionally, the results imply that a weak deck strengthened with a thin UHPC overlay will respond similarly to a deck composed of much stronger normal concrete. Test results were compared with ACI 318 predictions and showed the calculations are conservative for the slabs, underpredicting by 18-33%. The tensile capacity of the UHPC plays a large role on the overall strength and stiffness of a slab subjected to negative moment and the tensile strength should be included in capacity calculations, as recommended by FHWA. Overall, the results are promising and shed light on how a UHPC overlay may contribute to the overall strength of an existing bridge deck if implemented in a future overlay project in Montana.

17. Key Words Bridges; Bridge Construction; Bridge decks; Materials; Compressive strength; Fiber reinforced concrete; Concrete bridges; Highway bridges; High performance concrete; High strength concrete; Materials adaptation and use; Materials tests; Concrete overlays; Concrete tests; Tensile properties; Tensile strength; Thixotropy; Ultra high performance concrete		18. Distribution Statement No restrictions. This document is available through NTIS, Springfield, Virginia 22161.	
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UNIT CONVERSIONS

Measurement	Metric	English
Length	1 cm	0.394 in
	1 m	3.281 ft
	1 km	0.621 mile
Area	1 cm^2	0.155 in^2
	1 m^2	1.196 yd^2
Volume	1 m^3	1.308 yd^3
	1 ml	0.034 oz
Force	1 N	0.225 lbf
	1 kN	0.225 kip
Stress	1 MPa	145 psi
	1 GPa	145 ksi
Unit Weight	1 kg/m^3	1.685 lbs/yd^3
Velocity	1 kph	0.621 mph

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1 Introduction

1.1 Motivation

Ultra-high performance concrete (UHPC) has mechanical and durability properties that far exceed those of conventional concrete. However, using UHPC in concrete applications has been cost prohibitive, with commercially available/proprietary mixes costing approximately 30 times more than conventional concrete. Previous research conducted at Montana State University (MSU) has focused on the development and evaluation of non-proprietary UHPC mixes made with materials readily available in Montana. These mixes are significantly less expensive than commercially available UHPC mixes, thus opening the door for their use in construction projects in the state. The focus of the completed research was to investigate the use of UHPC for overlays in Montana Department of Transportation (MDT) bridge projects, and to summarize existing projects and specifications to assist MDT with project implementation.

Bridge deterioration, including decks and other members is a problem across Montana. UHPC overlays and patching/repairing may be a viable alternative to bridge/element replacement. Overall, the research performed herein was a required step to fully understand and capitalize on the benefits of using UHPC as a bridge deck overlay material, and to ultimately increase the lifespan of Montana's existing concrete infrastructure.

1.2 Background

UHPC became commercially available in the U.S. in 2000, and since then has been actively promoted by the Federal Highway Administration [1-6]. UHPC is generally understood to be a concrete with compressive strength of at least 20 ksi, post-cracking tensile strength of at least 0.72 ksi, and a discontinuous pore structure that improves durability by limiting permeability. These properties are achieved with: (1) low water-to-cement ratios, (2) aggregate gradations optimized for high particle packing density, (3) high quality aggregates and cements, (4) supplemental cementitious materials, (5) high particle dispersion during mixing, and (6) the incorporation of steel fiber reinforcement. Although the initial cost of UHPC far exceeds conventional concrete mixes, the use of UHPC has been shown to reduce life-cycle costs [7], as the increased durability of UHPC results in a longer service life and decreased maintenance costs. Further, the use of UHPC results in smaller/lighter structural elements.

Previous research conducted at MSU [8, 9] has included (1) the development of nonproprietary UHPC mixes that are significantly less expensive than commercially available mixes and are made with materials readily available in Montana (the mix has been designated MT-UHPC), (2) an investigation into several items related to the field batching of these mixes, (3) an exploration into the potential variability in performance related to differences in constituent materials, (4) the investigation of rebar bond strength and the subsequent effect this has on development length, (5) an investigation on the effects of varying the mixing process, batch size, and mixing and curing temperatures, including the development of a maturity curve, (6) the use of MT-UHPC for precast pile cap joints and shear keys between precast deck elements on two bridges spanning Trail Creek on Highway 43, west of Wisdom, MT, and (7) the investigation of making a thixotropic version of the MT-UHPC mix. This previous research has been successful and has clearly demonstrated the feasibility of using MT-UHPC in Montana bridge projects.

Some examples of other research conducted to explore applications of UHPC in the U.S. include bridge pier seismic strengthening [10], 100% UHPC structural elements such as girders [11], composite slabs [12], and even precast applications [13]. Considering the aging infrastructure in the country, two of the more promising applications of UHPC are its use in thin-bonded overlays for bridge deck rehabilitation [14-17] and bridge member repairs, rehabilitation, and structural patching [18-22]. In the overlay application, UHPC not only provides enhanced structural performance, it also provides protection from chloride penetration and water ingression [15]. The literature review in Chapter 2 dives further into the background of UHPC and focuses on these other applications for bridge repair.

1.3 Scope and Research Objective

The focus of this research was to explore potential applications of UHPC beyond its use in precast longitudinal joints and pile to pile-cap connections, and to conduct any required additional testing to ensure its successful use in these new applications. The research project began with a literature review into applications of UHPC for bridge repair, primarily focusing on thin-bonded overlays and bridge member repair. After the literature review was complete, the decision was made to primarily focus on the use of UHPC as a bridge deck overlay material.

The overarching goal of the remainder of the project was to investigate areas that need further development before fully providing MDT with the necessary information to successfully implement a UHPC bridge deck overlay on a bridge project in Montana. Specifically, the research conducted for this project focused on the following three objectives: 1) testing bond strengths to regular concrete and other material level properties of two non-proprietary UHPC mixes and one proprietary UHPC mix, 2) summarizing existing UHPC bridge deck overlay projects and specifications from other states to assist MDT in developing their own specification, and 3) construction and flexural testing of five different slabs to compare the effects of overlay thickness, substrate concrete strength, and negative vs. positive moment behavior.

It should be noted that the slab overlay testing portion of this research focused primarily on using proprietary commercially available UHPC (Ductal) rather than the nonproprietary MT-UHPC. This decision was made due to several limitations of the MT-UHPC. Specifically, a thixotropic version of the MT-UHPC has not been fully optimized, and the batch sizes of the MT-UHPC are limited to around 3-4 ft^3. Future research will look to overcome the limitations of this material.

1.4 Organization

There are seven chapters in this final report. Chapter 2 consists of a literature review into previous research related to UHPC including UHPC bridge deck overlays and bridge repair using UHPC. Chapter 3 describes and discusses the material testing performed to determine the potential of three different UHPC types for concrete repair. Chapter 4 presents the additional research into other state's existing UHPC specifications, completed UHPC overlay projects, and implementation issues and construction considerations for UHPC overlays. Chapter 5 documents the design, construction process, and structural testing of five UHPC overlay composite deck slab specimens. Chapter 6 discusses the results from the slab testing. Finally, Chapter 7 presents an overall summary of the project and discusses the specific conclusions drawn.

2 Literature Review

The literature review focused on applications of UHPC for bridge repair methods, with primary focus being on UHPC bridge deck overlays.

2.1 UHPC Bridge Deck Overlays

This section summarizes research on the use of UHPC as a bridge deck overlay. Specifically, this section discusses research projects conducted at Iowa State University, New Mexico State University, Missouri University of Science and Technology (Missouri S&T), and Montana State University.

2.1.1 Iowa State University & The Mud Creek Bridge

The first use of UHPC as an overlay in the United States was completed in 2016 on the Mud Creek Bridge on Buchanan County Road D48 near Brandon, Iowa [23]. This bridge is 102 ft long and 30 ft wide, is a continuous concrete slab bridge with two lanes, and has a 5% superelevation. Typically, UHPC is self-consolidating and therefore its use with superelevation is problematic. To accommodate this superelevation, a special thixotropic Ductal UHPC mix was produced by Lafarge Holcim by using thickening admixtures. Prior to the bridge application, Iowa State University performed a variety of tests to verify the performance and characteristics of this UHPC as an overlay material. These tests included prismatic slant shear tests and flexural tests, both on specimens with varying surface roughness to characterize the bond strengths. Figure 1 shows an example of a prismatic slant shear test, while Figure 2 shows a typical flexure test performed in this research. From the slant shear tests it was found that a minimum surface roughness of 0.125 in. gave the desired composite action, with the resulting failure occurring in the normal concrete (NC) layer. From the flexural tests, it was determined that a surface roughness of 0.25 in. resulted in the highest bond strengths, and the failure occurred in the NC.

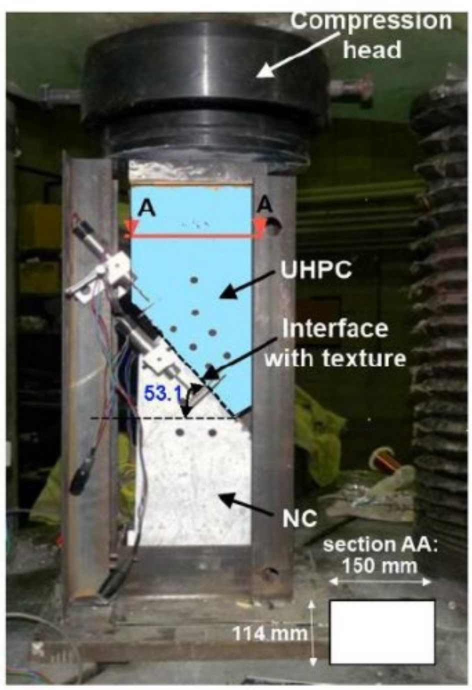


Figure 1: Example slant shear test setup [23]

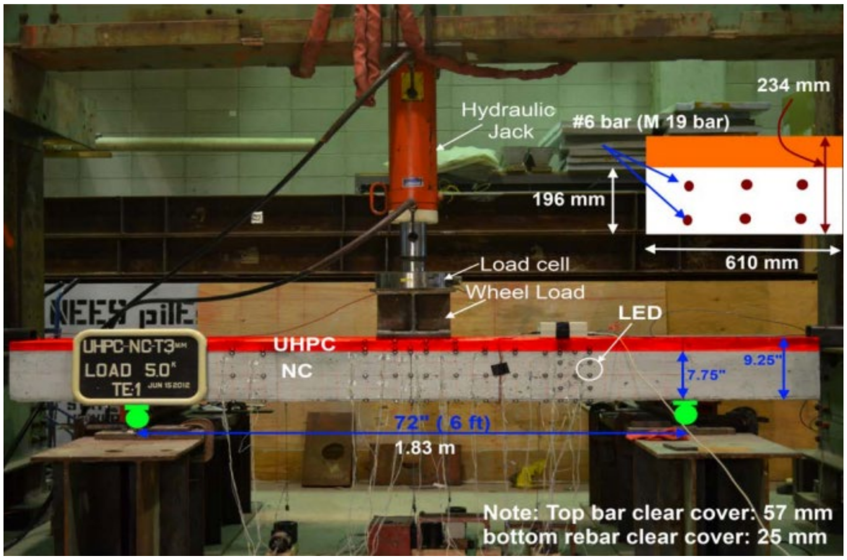


Figure 2: Example flexural test [23]

After the initial phases of this research, the thixotropic UHPC mix design was ready for field implementation on the Mud Creek Bridge. For this project, the top 0.25 in. of the deck surface was first removed, and the deck was then grooved along the bridge length with an amplitude of roughness ranging from one twelfth of an inch to one eighth. All batching and placing of the UHPC was performed on site by the contractor. A pair of high-shear pan mixers were used to mix the concrete. Each mixer had the capacity to mix 0.65 yd^3 (17.55 ft^3) of material. Loading and batching of the UHPC took approximately 20 minutes per batch. An overlay thickness of 1.5 in. was compacted and maintained by using a vibratory truss screed. All the mixing was done at one end of the bridge and transported using a mini concrete dumper. Grinding and grooving of the UHPC deck surface took place 4 days after placement (Figure 3), at which point the compressive strength had reached 12.3 ksi. Finally, the deck was evaluated using pull-off tests to quantify the bond strength between the UHPC and the substrate material.

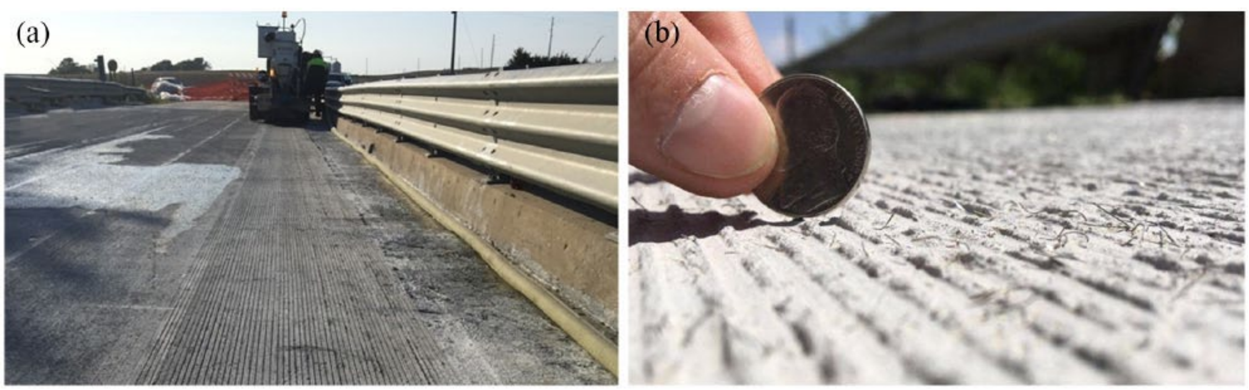
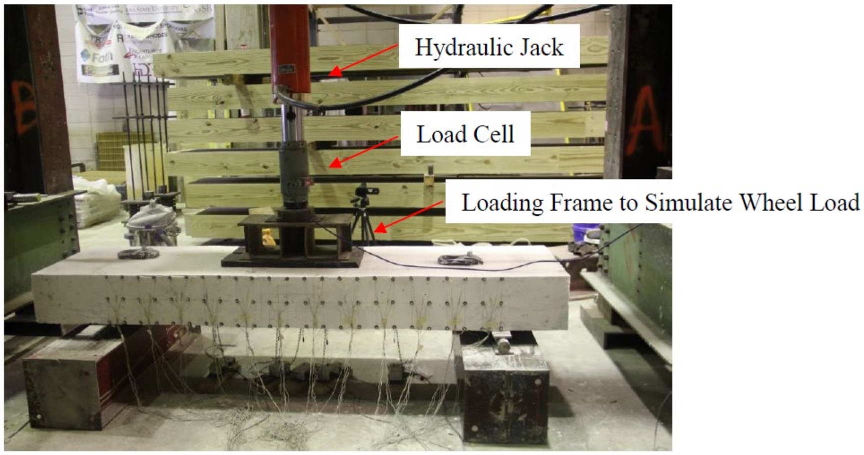


Figure 3: UHPC overlay on Mud Creek Bridge deck: (a) grooving of the surface; (b) closeup of finished surface [23]

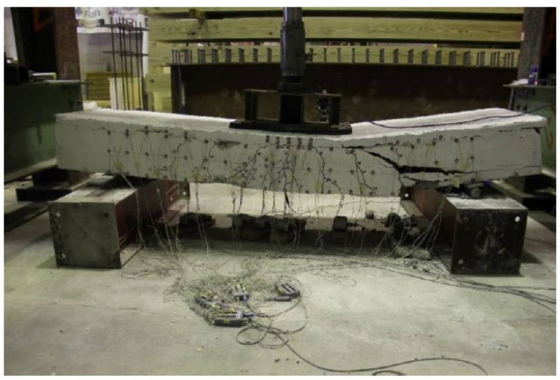
Additionally, the area over the pier locations was reinforced with a welded wire mesh to analyze the benefits of including this reinforcement in negative moment regions. After the construction was completed, a series of destructive and non-destructive tests were performed to ensure adequate bond strength between the UHPC and NC interface. Thermal imaging and the chain drag method were used first to identify eight potential delamination areas. Two out of the eight potential locations were then tested further using pull-off tests in accordance with the American Society of Testing and Materials standard (ASTM), ASTM C1583. Three additional areas that were determined to have good bond strengths were also tested for comparison. All tests resulted with failures in the NC layers, proving there was adequate bond strength between the NC and UHPC.

Two additional composite slabs with a wire mesh were cast using the UHPC mix from the bridge. This was done as a continuation of the flexure tests described earlier, but was focused on investigating the strength gain from using a wire mesh. The specimens were prepared similar to the negative moment sections of the bridge, though had to have the surface manually grinded instead of mechanically grooved like the bridge, due to the specimen size. The slabs being prepared are shown in Figure 4. The slabs were tested with a positive and negative moment and compared to an NC slab without an overlay. Results showed that both overlay specimens (positive and negative moment) showed increased strength and stiffness compared to the NC slab, though this was in part due to the additional 1.5-in. thickness from the overlay. The researchers concluded that the wire mesh did not add significant strength to the negative moment strength, due to the small amount of reinforcement added. A larger area of steel could lead to increased strengths, although could also affect the bond strength between the two layers. Figure 5 shows (a) the typical load setup used in this testing (similar to Figure 2), (b) the observed shear failure mechanism followed by partial UHPC debonding, and (c) the UHPC layer pried open after the test.

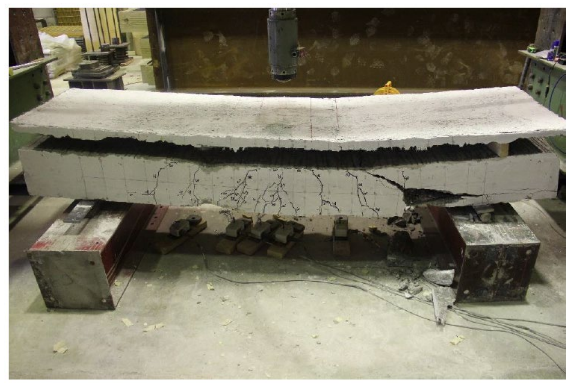
Figure 4: Casting UHPC on slab specimens [23]



a) Example concrete deck with UHPC overlay test setup



b) Partial overlay debonding following shear failure



c) UHPC layer pried open after the test

Figure 5: Observed failure of UHPC overlay flexural specimens [23]

2.1.2 New Mexico State University and Bridge 7032

New Mexico State University recently worked with the Transportation Consortium of South-Central States to create their own nonproprietary UHPC mix and evaluate its potential application as a bridge deck overlay [24]. The mix had an average compressive strength of 17.8 ksi. No comment was made about the flow of the proposed mix design. A series of tests were performed to determine the bond strength between the UHPC and NC layers. The tests included slant shear, split cylinder, split prism, and direct tension. Specimens were prepared by either grinding, adding horizontal grooves, adding cross-hatched grooves, or leaving a rough surface with a depth of 0.11 in. All the surface preparation techniques showed adequate strengths for the split cylinder and split prism tests, with strengths over ACI's recommendation of 150 psi for concrete repair [26]. However, none of these surface preparation techniques provided adequate strengths for the direct tension test. An additional chipped surface (not using a grinder) with a texture depth of 0.04 in. was tested after the completion of the original tests, and this method did provide adequate strengths. It was determined that the chipping method provided higher strengths because it did not plug the pores that help create the bond; however, this method is not recommended as it can damage the substrate layer more than expected. Overall, the bond assessment tests resulted in adequate bond strengths with textures (grinded or chipped) less than 0.08 in., which is lower than the minimum acceptable texture depth under field conditions of 0.25 in. from the American Concrete Institute (ACI) for conventional concrete repair [26], highlighting UHPC's high potential in overlay applications.

Both early-age and longer-term shrinkage was then investigated. A 6x6x24 in. beam was used to measure early-age shrinkage. It was found that 55% of shrinkage strain occurred while the UHPC was still plastic, and therefore this shrinkage strain would not cause a significant amount of horizontal shear stress between the UHPC overlay and the underlying NC deck. A 3x4x16 in. UHPC beam was used to measure long-term shrinkage. The beam was cured in a wet room for the first 7 days, then removed and cured in ambient conditions for the remainder of the 28 days. The shrinkage plateaued in the cure room around day 4. Outside the cure room, the shrinkage plateaued around day 20 with a max of about 450 μstrains.

To test the combined effects on shrinkage of composite UHPC-NC slabs, seven composite slabs were made with varying thicknesses of NC, exposure condition, steel reinforcement, and application of overlay. The NC slabs were cast first and at day 30 the surfaces were prepared with air hammers and chisels until the aggregates were exposed. Texture depth was measured following ASTM E965 and the average depths ranged from 0.06 in. to 0.15 in. Next, the UHPC overlays were cast and two photographs of the slab preparation are shown in Figure 6. Strains and external temperatures were measured over time. The results showed that the reinforcement of the NC layer had the greatest impact on reducing shrinkage caused by the UHPC overlay. However, NC layer thickness also played a role as thicker substrate slabs experienced more shrinkage than thinner slabs, with the same amount of steel; therefore, reinforcement ratio is key. Comparing the laboratory and outdoor exposure conditions, as expected, more uniform shrinkage was found for the laboratory specimens.

a) Placing the NC substrate b) Finishing the UHPC overlay

Figure 6: Preparation of UHPC-NC slabs [24]

To evaluate the effects of strengthening a beam with UHPC, fatigue tests were first performed on a plain channel girder, which was then overlayed with UHPC as shown in Figure 7. The girder was first fatigue tested through 1,000 load-unload cycles to an approximate mid-span deflection of 0.4 in. and average load of 20.3 kips. After unloading the beam, a residual mid-span deflection 0.0516 in. remained. A 1.0 in. UHPC overlay was then added to the beam and the same loading cycles were repeated. After the addition of the overlay, the girder saw an increase in flexural strength and required an additional 5.45 kips to reach the same deflection. After unloading the beam, a residual mid-span deflection of 0.037 in. remained, though no visual cracking or debonding was observed. The beam was then loaded to failure. Cracking first occurred at a deflection of 0.53 in. and a load of 33.1 kips. The ultimate deflection and load were 5.99 in. and 90.7 kips, respectively. Even at ultimate loading, little to no cracking occurred in the UHPC layer with only isolated locations of delamination.

Figure 7: Channel Girder with UHPC overlay [24]

After the non-proprietary mix was confirmed to be viable for use as a bridge deck overlay, a Bridge 7032 in Socorro, New Mexico was selected for an implementation project [25]. The bridge is approximately 300 ft long, 54 ft wide, consisting of two-lanes with a center median, four-spans, and made off multi-cell box girders as show in Figure 8. Damage consisted of potentially full depth transverse cracks at the negative moment regions over the column bents, though there was no evidence of delamination.

Figure 8: Bridge 7032 [25]

The proposed repair method consisted of removing the existing deck, installing a high-performance deck (HPD) layer, followed by the installation of a 1-inch thick UHPC overlay. The existing deteriorated concrete, expansion joints, existing metal railing, and the deck overhang were all removed to where the original rebar was exposed. Four mockup placements over the span of a year were conducted to get the NMDOT and contractors familiar with batching, mixing, and placing the UHPC overlay material. Bond tests with a HPD substrate were performed on the third and fourth mockups to ensure adequate bond. Additionally, two larger high-energy horizontal shaft mixers with a 1.0 yd^3 capacity were tested on the fourth mock-up to increase batch size and accelerate construction. A max batch size of 0.77 yd^3 was determined and was successfully placed an a HPD slab. It should also be noted that inadequate bond strengths and cracking were observed where the substrate was not adequately saturated as well where the UHPC overlay was not immediately covered after casting.

A standard slump test (ASTM C143) was used to determine the workability of the UHPC despite the common practice of performing a dynamic flow table test. Target slumps were between 8 and 10.5 inches as shown Figure 9. Additional testing included compressive strengths in accordance with the British Standard (BS) 1881 using cube samples at 2, 4, 7, 14, 28, and 56 days; flexural strength in accordance with ASTM C1609 at 7 and 56 days; and direct tension pull-off tests (no ASTM listed). Due to a variety of factors including lack of water content control and poor plastic covering on the steel strength molds, average 56-day compressive strengths were only 13.6 ksi.

a) Slump Measurement

b) Spread Measurement

Figure 9: Material Consistency Measurements

As described previously, construction of the new deck began with removal of the deteriorated and a volumetric replacement with HPD. The top surface was textured using a tine rake with a minimum depth of 0.25-inch. The cured surface was then ceramic bead blasted and kept saturated up until the placement of the UHPC overlay. The UHPC was placed over a total 105 batches across four placement days. UHPC was delivered using a concrete buggy, then spread and vibrated by hand, followed by a vibratory screed to finish and maintain the thickness of the UHPC overlay. The first placement immediately started cracking because it was not covered under plastic sheeting and burlap after surface finishing. The remaining placements required an immediate application of a curing compound and then plastic sheeting as soon as possible. 90-

degree cold joints were placed over a positive moment region when construction was stopped between placements.

Throughout the construction of the bridge, a variety of thermocouple, steel reinforcement strain gauges, and concrete strain gauges were internally embedded to monitor the structural performance of the superstructure, HPD layer, and UHPC overlay. All sensors were wired to a multiplexor connected to a solar powered datalogger to continuously monitor the performance.

To test the overlay-substrate bond strength, hammer sounding, chain drags, and thermal imaging were performed to determine 10 areas of potential delamination. Direct tension pull-off tests were then used to determine the bond strength. The average bond strength was 239 psi, slightly below the ACI recommended of 150 psi, however, 4 the cores failed in the epoxy.

2.1.3 Missouri S&T

The Missouri Department of Transportation funded a research project at Missouri S&T on designing an optimized UHPC mix for bridge deck overlays [27]. Sixteen NC slabs were prepared to test different mix designs and thicknesses and compare to latex modified concrete (LMC). The slabs contained rebar mats near the top and bottom of the slab. The top surface was prepared using a chemical surface retarder and a stiff brush to expose the aggregate. The slabs were then left outside for 12 months prior to applying the overlay. Strain gauges, relative humidity sensors, and thermocouples were embedded between NC and UHPC layers (Figure 10). A life cycle cost analysis was also performed to compare UHPC and NC. The results showed that a 1.0 in. UHPC overlay was the most cost-effective based on deterministic and probabilistic results. Additionally, the cost of UHPC is likely to decrease as demand and production increases, making it more desirable as an overlay material in the future.

Figure 10: Example NC slabs with UHPC overlay and instrumentation [26]

2.1.4 Thixotropy for MT-UHPC

After investigating the use of UHPC as a bridge deck overlay material, there was an apparent need to adjust the current MT-UHPC mix design to exhibit thixotropic behavior. A thorough search was conducted on the topic of UHPC mix adjustments for thixotropy with little results, because all UHPC overlay implementation projects have used proprietary mixes. Montana State University [28] explored the following two potential methods for creating a thixotropic mix: 1) increase the amount of steel fibers and 2) use an additional admixture, and the overall results are summarized herein. The standard steel fiber amount used for MT-UHPC is 2% by volume and this was doubled to 4% for the thixotropic study. The following two different Master Matrix viscosity changing admixtures were tested: VMA 358 and UW 450. Overall, results showed that neither doubling the steel fibers nor adding the VMA 358 admixture yielded the desired effects on the overall viscosity of the MT-UHPC. However, the mix using UW 450 exhibited the desired increase in viscosity and was able to maintain its shape on a 6% slope. The increase in viscosity led to poor consolidation in the test cylinders, which in turn led to below average compressive strengths of only 13.58 ksi at 28 days. With respect to adjusting the MT-UHPC mix to exhibit thixotropic behavior, the initial results are promising. However, further research may be required to see how the thixotropic mix can be batched in a larger pan mixer, and if better consolidation in test cylinders will lead to higher strengths.

2.2 Bridge Repair Using UHPC (University of Connecticut)

This section summarizes research conducted at the University of Connecticut that explores using UHPC to repair existing bridge elements [29-31]. More specifically, they investigated a method for repairing the ends of steel girders using UHPC. This research consisted of three phases, which will be described in detail below.

2.2.1 Phase I – Rolled Girder Testing and FE Modeling

Phase I of their research focused on creating a UHPC repair method for the ends of deteriorated steel girders by testing three half-scale, rolled girders with varying amounts of deterioration [30]. The proposed repair method involved welding studs around the damaged portion of the web and flange, and then encasing the studs in UHPC. The three girders tested in this research consisted of an undamaged girder, a damaged girder, and a repaired girder. A 14-ft long W21x55 girder was selected as the test girder size because it was approximately half the scale of a W36x160 bridge girder commonly used in Connecticut and it has the same web slenderness ratio. To simulate corrosion in the two deteriorated test girders (damaged and repaired), the end of the lower tee of the girders were removed using a plasma cutter. A portion of the web and flange were then milled off using a computer numeric controlled (CNC) milling machine. Full penetration groove welds were then used to attach the lower tee section back to the girder for the damaged and repaired specimens. The tee for the repaired section was not attached until after the studs were welded on. Nelson Stud Welding H4L Headed Concrete Anchors, 0.375 in. diameter by 1.25 in. long, were used as scaled down equivalents of the standard 0.75 in. size used in typical composite steel and concrete decks. The studs were staggered vertically and horizontally on opposite sides of the web to avoid bearing stress concentrations. The studs were then encased in a 25 in. long, 13 in. high, 1.75 in. deep, panel of UHPC. The stud pattern and UHPC panel are shown in Figure 11. The UHPC panel was formed using R-10 foam board and plexiglass. The girder and formwork were coated in mineral oil prior to casting to represent the lack of bond strength with girder paint. Mineral oil was not applied to the studs. A JS1212 Ductal mix provided by LaFarge Holcim was used for the UHPC.

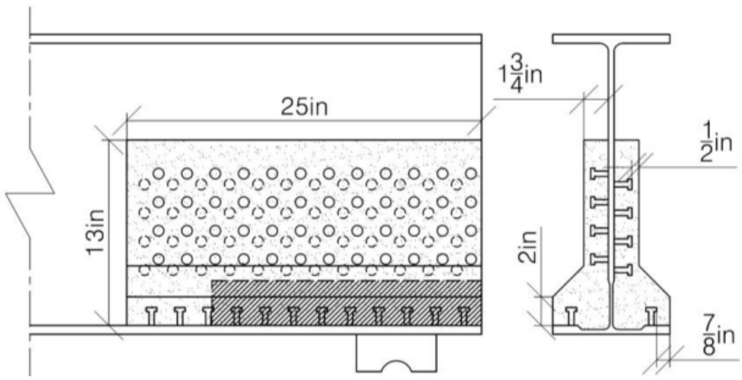
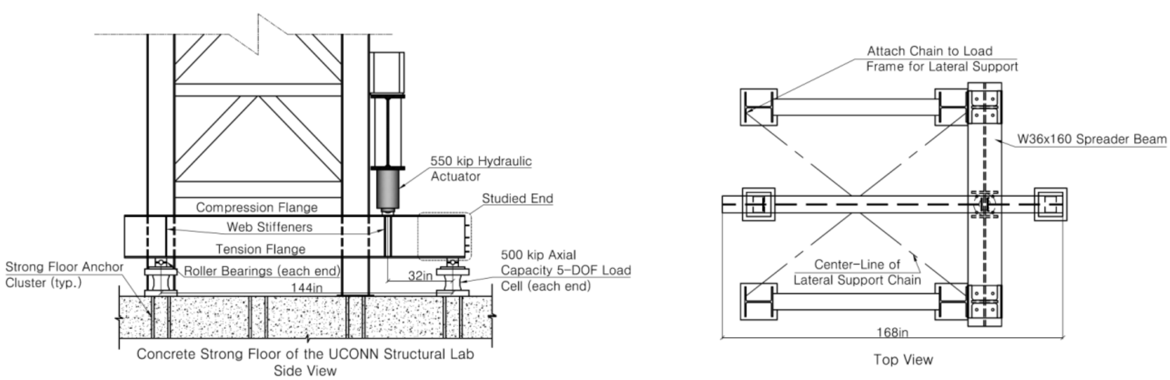


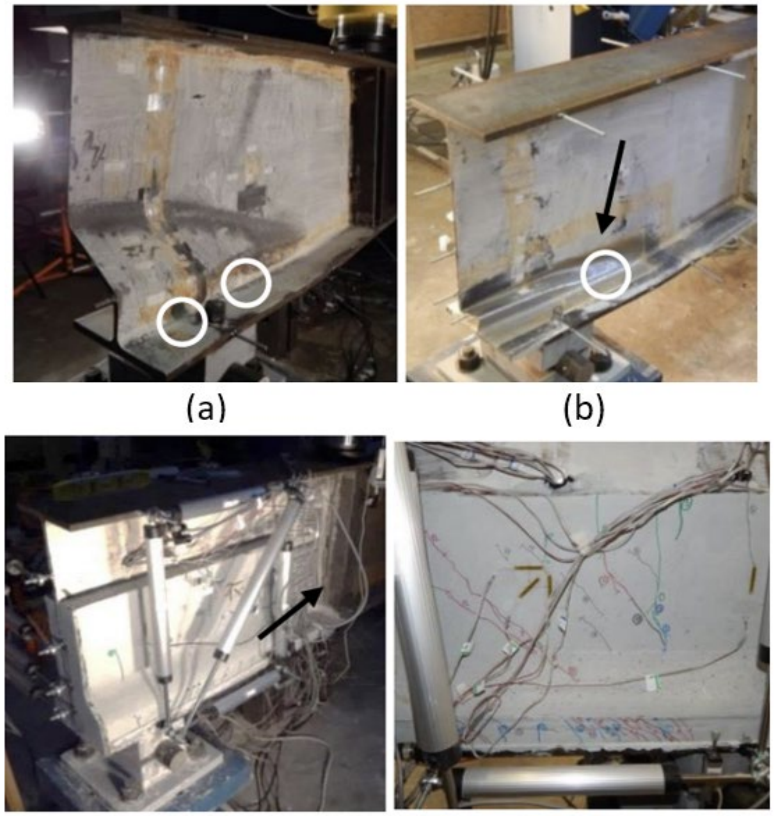
Figure 11: UHPC Panel for the Large-Scale Repair [30]

All three girders were tested using the same loading setup shown below in Figure 12. Web stiffeners (0.5 in.) were added at the loading point and the end not being tested, but not to the testing end. Additionally, the top of the beam was laterally restrained near midspan using clevises and chains. The web of the girder near the testing end was also treated with limestone and water to produce a thin white layer that would flake off during straining of the web and highlight any damage accrued during testing. The undamaged girder failed at a load of 180 kips through web buckling at the end of the girder, over the entire height of the bearing. The damaged girder failed at a load of 43.4 kips due to instability in the web at the top of the damaged section. The repaired girder reached a maximum load of 230 kips, where it experienced extensive flexural yielding but did not fail. All three girders after testing are shown below in Figure 13. The repaired girder was able to hold over five times the capacity of the damaged girder, and over 28% of the undamaged girder. All three girders had similar stiffnesses.



a) Side View b) Top View

Figure 12: Large-Scale Experiment Setup [30]



(a) (b) (c) (d)

Figure 13: Conditions of each specimen after testing (a) Undamaged, (b) Damaged, (c) Repaired, (d) UHPC Cracking [30]

A finite element (FE) model was then created to predict the capacities of steel girders repaired using a variety of UHPC strengthening details. The model was first compared to the results from the half-scale tests to validate its accuracy, then it was used to evaluate several UHPC repair methods not tested in this research to evaluate their efficacy. The model was shown to accurately predict the failure modes and shapes; however, it potentially predicted inaccurate stress concentrations making the model conservative. Eight new repair methods were then evaluated across three types of girders with the FE model. The eight repair methods were created for full-height, half-height, and L-shaped repair; and the three girders evaluated were rolled girders without stiffener, rolled girder with stiffener, and a plate girder. All repair methods were shown to increase the capacity compared to the undamaged girder.

2.2.2 Phase II – Stud Testing and Model Improvement

To improve the accuracy of the FE model, Phase II of the research evaluated the effectiveness of various repair details (e.g. stud layout, concrete cover, underlying steel condition) [29]. To test the strength of the studs on older degraded steel, smaller scale push-off tests were performed on rolled steel girders that were salvaged from an old bridge. For the push-off tests, studs were welded to the web section of the salvaged girders and then encased in UHPC as shown in Figure 14. Eight specimens were made by varying the stud size, layout, spacing, concrete cover, and concrete type (UHPC and NC). The same JS1212 Ductal UHPC mix provided by LaFarge Holcim from Phase I was used for all UHPC specimens. The specimens were compressed with one end bearing on the girder and the other end on the concrete to analyze the stud failure. The test setup is shown in Figure 15.

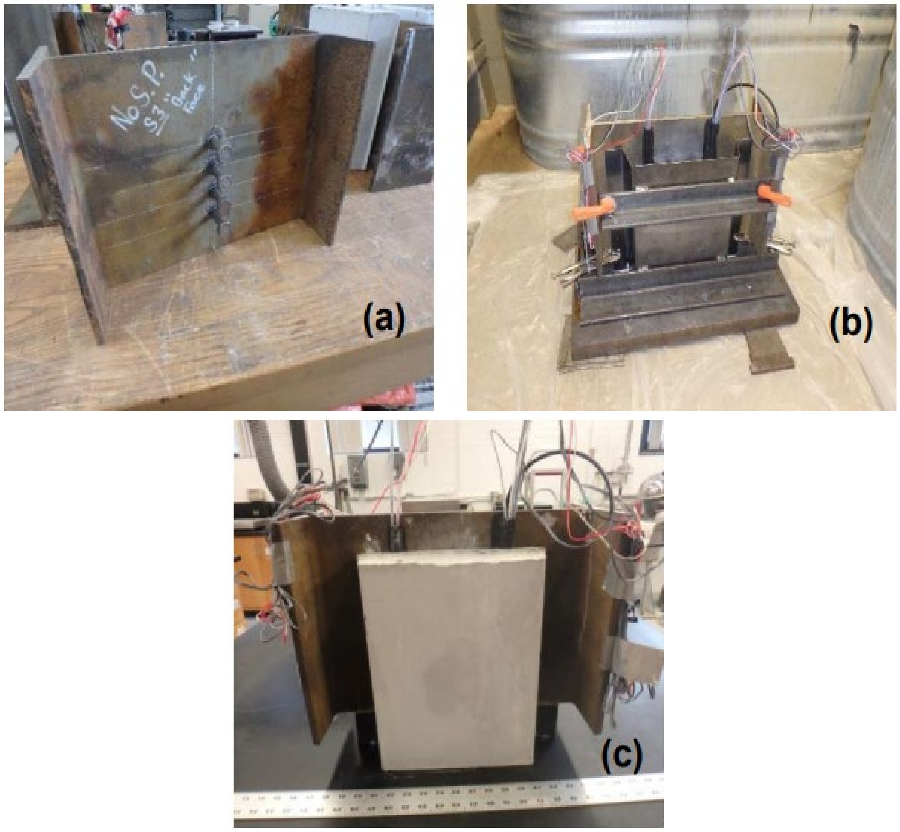


Figure 14: (a) Beam prior to casting; (b) beam with formwork used for casing concrete, and (c) completed push-off sample [29]

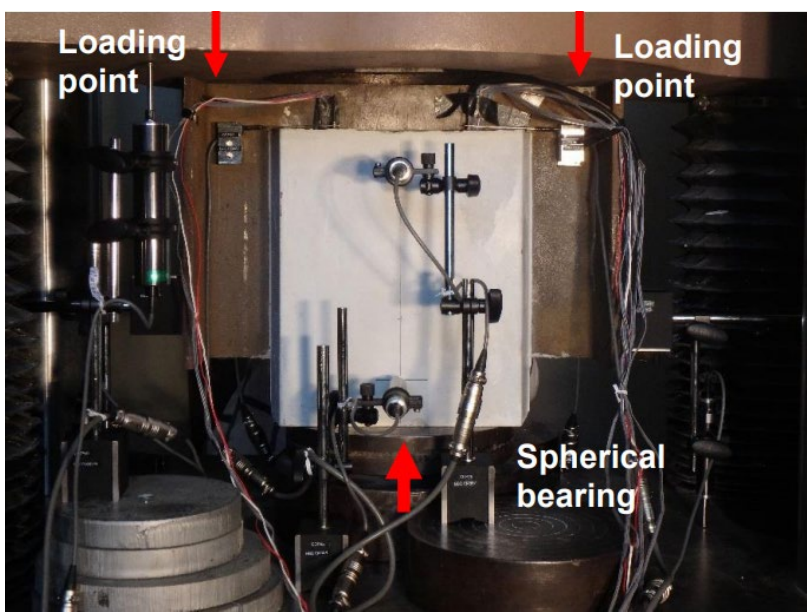


Figure 15: Typical experimental setup for push-off test [29]

The NC specimen was deemed unsuitable for the envisioned repair method as it split in tension when larger loads were applied. Conversely, all UHPC specimens failed through shear failure of the stud shank, with little to no cracking in the UHPC, and exceeded the theoretical capacity outlined by AASHTO [32]. Results showed that the 0.5-in. diameter studs performed the best, with the 0.675-in. diameter studs having a 25% reduction in capacity when compared to the 0.5-in. diameter studs. The changes in layout and spacing had little effect on the overall strength of the capacity of the specimens. Three-dimensional scans were also used to more accurately model the corroded girders in the FE model. A 3D scanner was used to create a point cloud of the corroded girder specimens, the point cloud was imported into an FE model, and ultimately, the methodology was shown to accurately model the corroded section and stress concentrations.

2.2.3 Phase III – Full-Scale Repair and Testing

Phase III of the research was focused on applying the developed repair method on four full-scale plate girders [30]. The design of the full-scale plate girder was chosen to represent an average bridge in Connecticut. A permanent girder was designed with a splice near one end to allow for a section that could be connected to replaceable test panels for each tested girder specimen as seen below in Figure 16. Overall, the testing setup was similar to that of the girders tested in Phase I, though upscaled to account for the larger plate girders. Local suppliers provided and fabricated the steel girders using grade A36 steel plates. Corrosion was simulated using sand blasting to give a non-uniform section loss near the ends of the girder. Other methods such as electrochemical corrosion and CNC milling were also considered but did not produce as desirable of results compared to the sand blasting.

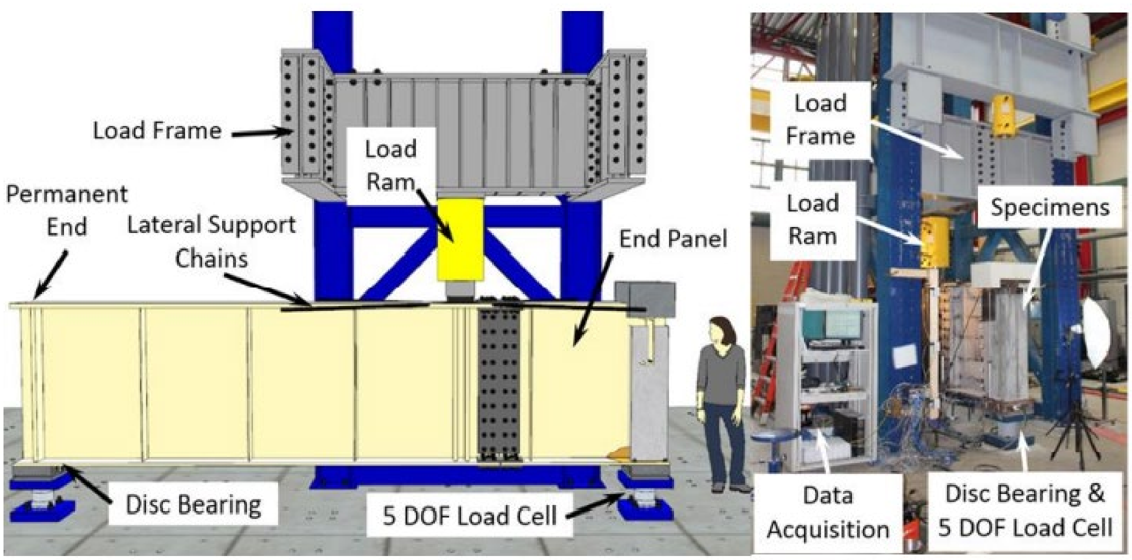


Figure 16: Testing setup [31]

Two full-height repairs (Full Height 1 and Full Height 2), one half-height repair (Half Height), and one baseline damaged girder were tested. The difference between the two full-height repairs were the stud layouts and UHPC used. Full Height 1 used Ductal JS1000, a slower setting and higher strength mix, whereas Full Height 2 used Ductal JS1212, a faster setting and lower strength mix that was also vibrated to simulate vehicle traffic on a bridge. Half Height used Ductal JS1212 (same as Full Height 2) and only covered the lower half of the web. The number of studs to be welded to the web were determined by dividing the estimated nominal shear capacity of the plate girder by the design shear capacity of a stud. This resulted in 28 studs being used in varying layouts for the three repaired specimens. The Full Height and Half Height repair drawings are shown in Figure 17.

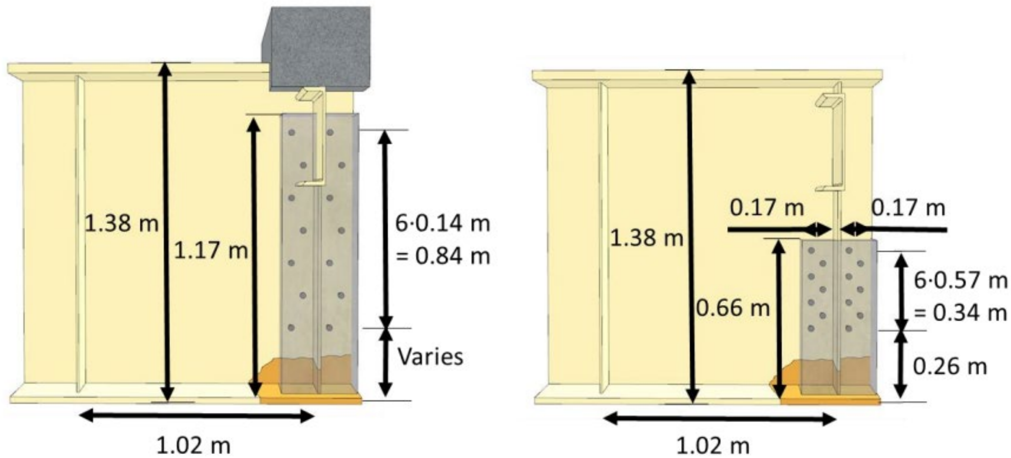


Figure 17: Full Height and Half Height Repairs [31]

The baseline corroded plate girder reached a flexural capacity of 95 kips and failed due to localized buckling of the web and bearing stiffener in the corroded region. The Full Height 1 repaired girder reached an initial peak capacity of 527 kips at a deflection of 0.504 in.; afterwards however, the girder was then able to sustain 450 kips up to a deflection of 0.994 in. where it failed due to web buckling of the end panel. Full Height 2 and Half Height performed very similarly to Full Height 1 with post-peak sustained load capacities at similar deflections. Full Height 2 reached initial peak and post-peak sustained load capacities of 497 kips and 450 kips, respectively, whereas Half Height reached respective capacities of 472 kips and 400 kips despite only covering half the cross section. The beams tested in this research were then modeled using the FE method developed and refined in the previous research phases. The predicted responses from this method were very similar to the measured responses from the tests. Based on the results, the half-height repair with 0.5-in. headed shear studs was recommended for use in repairs moving forward. Despite having a lower capacity, the method allows for easier construction and requires less UHPC. Additionally, the study concluded that although AASHTO's predicted values for shear stud capacity are conservative, they should still be used in order to account for uncertainties in weld quality.

2.3 Summary of Literature Review Findings

UHPC has been successfully used in several projects and across several applications, as described in this chapter. For bridge deck overlay applications, the literature review findings reveal that other state DOTs are almost exclusively using proprietary mixes during implementation projects. This is most likely a result of the same implementation issues being researched in the current work at MSU, including batch sizing for large volumes and the need for thixotropic behavior in this specific application. For example, the UHPC overlay implementation project conducted by Iowa State University used a special thixotropic Ductal UHPC mix produced by Lafarge Holcim. Additionally, adequate bond strengths have been found between UHPC and NC substrates across a variety of surface preparation techniques, with NC surface preparations below minimum depth requirements.

A repair method using shear studs and UHPC for the ends of steel girders was investigated in depth by the University of Connecticut. This method was shown to work well. Initial testing that compared an undamaged girder, a damaged girder, and a repaired girder, showed that while all three girders had similar stiffnesses, the repaired girder was able to hold over five times the capacity of the damaged girder and over 28% of the undamaged girder. Based on the comparisons made during the full-scale testing, a half-height repair with 0.5-in. headed shear studs was recommended, because although it has a lower capacity than a full-height repair, strengths are adequate, the method allows for easier construction, and less UHPC is required.

Overall, using UHPC as a bridge deck overlay was found to be a promising application for implementation on bridges in Montana, and after assessing the findings of the literature review, the research team and technical panel decided to move forward with pursuing UHPC bridge deck overlays for the remainder of the project. Additional research was required to evaluate available UHPC mixes for their viability in the application and eventually to assess the overlay's ability to strengthen a deck. Therefore, the next phase of this research, discussed in Chapter 3, was to evaluate material level and bond properties of several UHPC mixes to evaluate which material would be best to move forward with the larger-scale testing.

3 Material Level Evaluation

This chapter documents the material-level evaluation of multiple UHPC mixes. Specifically, this evaluation focused on surface preparations and the subsequent bond strengths between the UHPC and normal substrate concrete. Three different UHPC mixes were evaluated for the desired concrete repair/overlay application. For each of these mixes, the workability, compressive strength, tensile strength, and bond strength were investigated. The mixes in this research included MT-UHPC, MT-UHPC with the addition of a viscosity modifying admixture for thixotropy, and a proprietary thixotropic Ductal mix. The mix designs and constituent materials are first discussed, followed by a description of the testing program, and then finally followed by the results from these tests.

3.1 Materials

Three UHPC mixes were investigated at the material level to evaluate compressive and tensile strength, and the bond strength with substrate concrete. The mixes include MT-UHPC, MT-UHPC with the addition of a viscosity modifying admixture for thixotropy (designated here as MT-UHPC-T), and a proprietary thixotropic Ductal mix (designated here as Ductal-T). In this section, first the substrate conventional concrete is discussed, followed by a discussion of each UHPC mix. It is important to note that trial batches were performed for the MT-UHPC-T and Ductal-T mixes to determine admixture/water dosages; however, specific details on these trial batches are not included in this report.

3.1.1 Substrate Concrete

The substrate concrete mix was a conventional 4 ksi design strength mix targeting 3% air entrainment. The mix design for a 3-ft^3 batch is shown below in Table 1. The substrate concrete was mixed in a standard rotating-drum, fixed-vane mixer. The coarse and fine aggregate and approximately 4 pounds of the water were added first and mixed for 3 minutes. Once the aggregates reached saturated surface dry (SSD) condition, the air entraining admixture was added, and the aggregates were mixed for 2 additional minutes. The water and cement were then added simultaneously and mixed for approximately 8 minutes. A slump test was performed for each mix in accordance with ASTM C143 and an average slump of 2" was measured.

This concrete mix was used as the substrate concrete for the bond tests completed for each of the UHPC mixes tested in this research. These tests will be discussed in detail in a later section.

Table 1: Substrate concrete mix design for a 3 ft^3 batch

Item	Weight (lbs)
Water	37.6
MasterAir AE 200	13.67 (ml)
Cement	68.4
Coarse Aggregate	218.7
Fine Aggregate	116.2

3.1.2 MT-UHPC

The standard MT-UHPC mix was developed in previous research at MSU. The mix design for a 3-ft^3 batch is shown in Table 2. A fixed-drum, rotating fin high-shear mortar mixer (IMER Mortarman 360) was used to mix the MT-UHPC using the procedures developed in previous research. This procedure involved adding the fine aggregate and silica fume first and mixing for 5 minutes. Cement and fly ash were added next and mixed for an additional 5 minutes. The premixed water and HRWR were then added to the mixer. The mix took approximately 15 minutes to turn over and become fluid. The steel fibers were then added and mixed for 3 minutes. A static flow test was performed following ASTM C1856 and a flow of 10.25” was measured as shown in Figure 18.

Table 2: MT-UHPC mix design for a 3 ft^3 batch

Item	Weight (lbs)
Water	33.2
CHRYSO Fluid Premia 150 (HRWR)	7.2
Steel Fibers	29.2
Cement	144.4
Silica Fume	30.9
Fly Ash	41.3
Fine Aggregate	172.9

Figure 18: MT-UHPC static flow test

3.1.3 MT-UHPC-T

The MT-UHPC-T mix was identical to the standard MT-UHPC mix, with the exception of the viscosity modifying admixture. The mix design for a 3-ft^3 batch is shown in Table 3. The viscosity modifying admixture was MasterMatrix UW 450 (spec sheet included in Appendix A). A total of 15 fluid ounces of this admixture was used in the 3-ft^3 batch, which equates to a dosage rate of 6.9 fluid ounces per 100 lbs of cementitious materials (6.9 fl oz/cwt). A fixed-drum, rotating fin high-shear mortar mixer (IMER Mortarman 360) was used to mix the MT-UHPC-T, using a procedure similar to that used for the standard MT-UHPC. After adding the HRWR it took over 15 minutes for the mix to turn over. Once the fibers were thoroughly mixed, the MasterMatrix UW 450 admixture was added and mixed for 5 minutes. The static and dynamic flows were measured at 4.0” and 5.5”, respectively (Figure 19). The dynamic flow was slightly lower than desired; however, the consistency of the mix was appropriate, and the mix performed well. This was the first large-scale batch of a thixotropic version of MT-UHPC, and although some adjustments may be warranted to optimize the flows, the results are promising.

Table 3: MT-UHPC-T mix design for a 3-ft^3 batch

Item	Weight (lbs)
Water	33.2
CHRYSO Fluid Premia 150 (HRWR)	7.2
Steel Fibers	29.2
Cement	144.4
Silica Fume	30.9
Fly Ash	41.3
Fine Aggregate	172.9
MasterMatrix UW 450	15 (oz)

Figure 19: MT-UHPC-T static (left) and dynamic (right) flow test results

3.1.4 Ductal-T

Materials and mix proportions were provided by LafargeHolcim for the Ductal-T UHPC mix. The mix design for a 3-ft^3 batch is shown in Table 4.

Table 4: Ductal-T mix design for a 3 ft^3 batch

Item	Weight (lbs)
Water	31.8
F5 Admixture	4.1
Steel Fibers	46.5
Ductal Premix	375.0

Again, the IMER Mortarman 360 mixer was used to mix the material. The dry ingredients were added to the mixer first and mixed for 3 minutes to ensure that the mix was homogenized. The water was then added, and immediately followed by the F5 admixture. After 4 minutes of mixing, the mix began to turn over, and after an additional 3 minutes the mix had fully turned over and the steel fibers were added. The fibers were then mixed in for 3 minutes. An initial dynamic flow was measured at 6”. This was slightly lower than the desired dynamic flow of 6.25”-7.25” (as recommended by LafargeHolcim). An additional 1.35 lbs (already accounted for in Table 4) of water was then added and mixed for 2 minutes. A new dynamic flow test was performed and a flow of 6.5” was recorded (Figure 20). A static flow of 4” was also recorded.

Figure 20: Dynamic flow test results for Ductal-T

3.2 Experimental Design

This research consisted of testing the compressive, tensile, and bond strength of three UHPC materials. This section discusses details on the tests used to evaluate these properties. Specifically, general compressive and tensile test methods are discussed, followed by detailed descriptions of the direct-tension and slant-shear bond tests.

3.2.1 Compressive and Tensile Testing

Compressive strength testing was performed per ASTM C1856 and ASTM C39 for the UHPC and substrate concrete mixes, respectively. Compressive strengths for the UHPC materials were obtained at 7, 14, and 28 days, while compressive strengths for the substrate concrete were only obtained on the day that the direct tension and slant shear tests were performed. Flexural strength testing was performed at 28 days in substantial accordance with ASTM C1609 on 20”x6”x6” prisms. A typical flexural specimen in the load frame is shown in Figure 21.

It should be noted that these test specimens were prepared following procedures outlined in previous MSU research [1]. However, additional procedures were required to consolidate the thixotropic mixes. Specifically, these specimens were placed on a vibration table during casting.

Figure 21: Example flexural test performed on a Ductal-T specimen

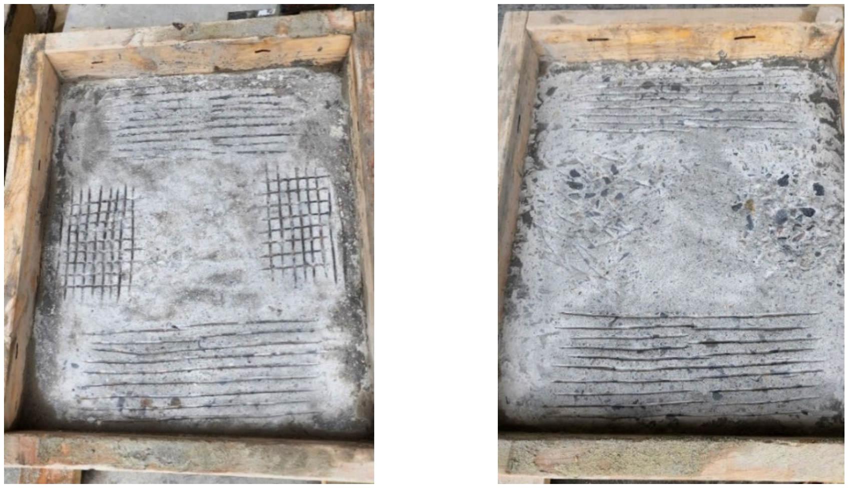
3.2.2 Direct Tension Testing

Direct tension testing was performed by following similar procedures outlined in ASTM C1583 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). This testing determines a limit on the tensile bond strength between standard concrete and the UHPC mixes and is dependent on the substrate concrete surface preparation. In this test, failures will typically occur either at the bond between the two materials, in the substrate concrete, or in the adhesive between the core and test fixture. This test is typically conducted in the field on in-place slabs by pulling directly on cores from the slab and recording the maximum pulling force. In this research, due to availability of equipment, small test slabs with UHPC overlays were constructed in the lab, and cores were extracted and tested in direct tension with an MTS compression/tension load frame.

The slab specimens were 23”x19.25” and were constructed first with 3” of normal substrate concrete (Figure 22). Two substrate slabs were constructed for each of the three UHPC mixes, for a total of six slabs. The substrate concrete slabs were cured in the cure room for at least 28 days. After curing, the surfaces of the slabs were prepared using an angle grinder. After first grinding the top surface flat, three different surface preparation techniques were explored to examine the efficacy of each of these methods. The first method, which is designated as typical (T) included parallel grooves in one direction that were ¼” deep and ⅛” wide and spaced at ½” intervals (Figure 23a and Figure 24a). The second method was designated as cross-hatch (XH), and consisted of grooves of the same size as those designated for T, but in both directions (Figure 23a and Figure 24b). The final method was designated as chipped (C) and consisted of a jack-hammered surface with an approximate roughness of ¼" (Figure 23b and Figure 24c).

It should be noted that the surface roughness achieved for the T specimens should yield conservative results, as surface preparation techniques used in the field are typically more aggressive than this, with a minimum specified texture depth of ¼” according to ACI recommendations for conventional concrete repair [26]. Therefore, the T specimens will provide for a conservative limit on bond strength, while the cross-hatch and chipped specimens will provide more data for discussion.

Figure 22: Typical substrate concrete slabs for direct tension specimens



a) T and XH b) T and C

Figure 23: Substrate surface preparations for direct tension testing

a) T Surface Preparation b) XH Surface Preparation c) C Surface Preparation

Figure 24: Close-up views of surface preparation methods

After preparation, 1.75” of UHPC was placed on top of the prepped slab surfaces. The substrate surfaces were typically wetted with a sponge prior to the placement of the UHPC. However, one of the Ductal-T slabs was not wetted prior to placement, which had a significant effect on performance, as will be discussed in a later section. After placement of the thixotropic UHPC mixes, the slabs were then consolidated by placing the slab on the vibration table and vibrating for several seconds while tapping with a rubber mallet (as shown in Figure 25).

Figure 25: Typical consolidation process for thixotropic specimens including shake table (located below specimen form) and external tapping with rubber mallet

After curing, the slabs were then cored to extract the direct-tension specimens (Figure 26). This coring was done using a Diamond Products Core Bore 748 drill, with a 2” inner diameter Husqvarna diamond core drill bit (Figure 26b). The cores were drilled through the slabs, and then cut to length. Typically, at least 1.5” of UHPC and substrate concrete was desired, though some samples were cut shorter due to a slightly thinner overlay. Overall, 11 successful core specimens were extracted for MT-UHPC (8T, 2XH, and 1C), 8 cores for MT-UHPC-T (6T and 2XH), and 11 cores for Ductal-T (8T, 2XH, and 1C). After extraction, the cores were then epoxied to two 2” diameter, 1” thick steel discs (one on each end) using Simpson Strong-Tie SET-XP epoxy (Figure 26d). Note that the slab in Figure 26c is in the same orientation as the surface preparations shown in Figure 23.

a) Cured slab before coring

b) Core drill in place on slab

c) Slab after coring

d) Core prepped for testing

Figure 26: Typical direct tension core specimen preparation

After preparation, the specimens were tested in an MTS compression/tension load frame, as shown in Figure 27. As can be seen in the figure, the test fixture consisted of a series of shackles and eyebolts to ensure proper alignment and alleviate any potential eccentricities introduced as a result of support fixity. The ultimate tensile bond strength was then calculated by dividing the ultimate load by the cross-sectional area of the specimen.

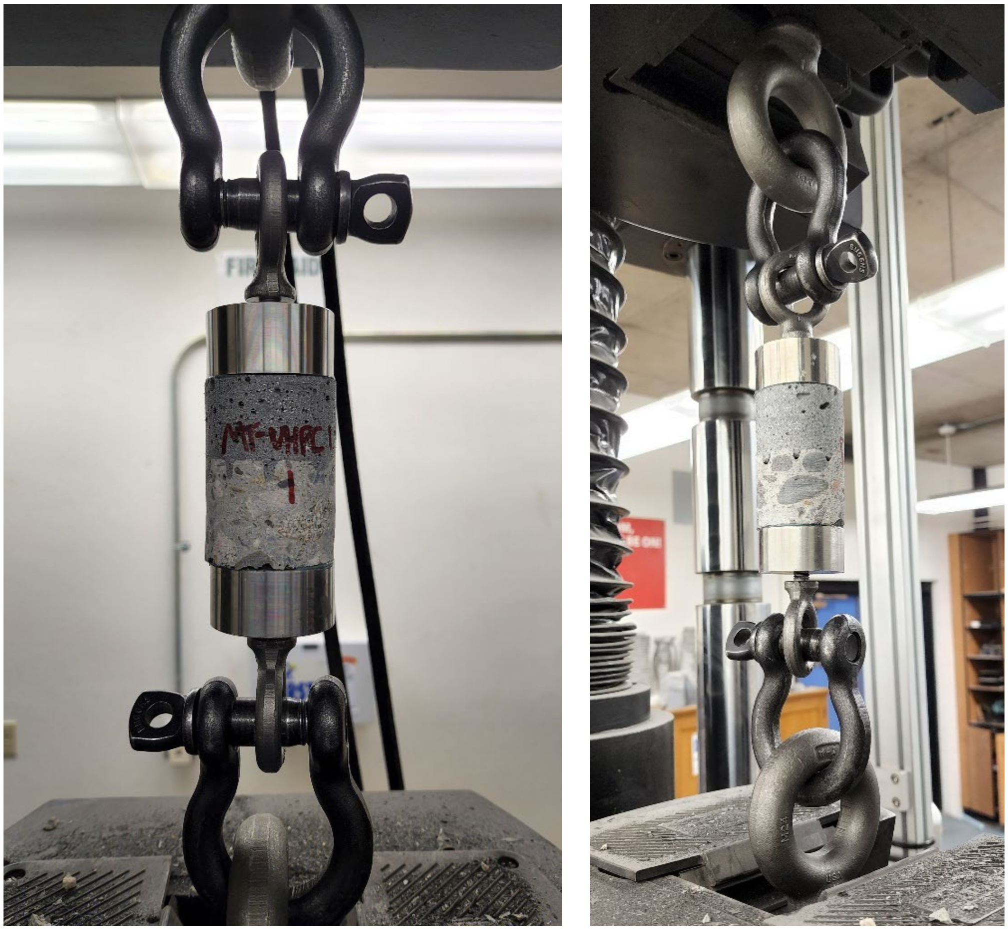


Figure 27: Example direct tension specimens prior to testing

3.2.3 Slant Shear Testing

Shear bond strength is a critical parameter needed to fully assess the bonding of UHPC to standard concrete for a range of potential applications. In this research, this property was tested with slant shear tests. These tests were performed in substantial accordance with ASTM C882 Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear. Typically, failures will occur either at the bond between the two materials or in the substrate concrete.

To adapt the ASTM standard for testing UHPC, 4”x8” cylinders were cast instead of the recommended 3”x6”. This was done to accommodate the size of the coarse aggregate in the substrate concrete and to allow for a larger surface area for preparation. For placement of the substrate concrete, wood forms were used to rotate the cylinders 30-degrees, as shown in Figure 28. After initial curing, the substrate concrete was removed from the molds and placed into the cure room. After at least 28 days, the samples were removed from the cure room and the top surface of the incline was grooved to simulate surface preparation that may take place prior to UHPC placement. The same “typical” surface preparation discussed for direct tension testing was investigated for slant shear. Specifically, an angle grinder was used to grind the top surface flat and apply grooves ¼” deep, ⅛” wide, at ½” spacing on the inclined surface (Figure 29a). To assess the worst-case scenario, the grooves were aligned parallel with the direction of the shear loading.

Figure 28: Typical substrate concrete half cylinders for slant shear specimens

After curing and surface preparation, the slant shear substrate concrete samples were then placed back into the 4”x8” cylinder molds in order to place the various UHPC mixes (Figure 29b). The top surfaces of the substrate concrete were wetted prior to placement of the UHPC. At 24 hours after UHPC placement, the cylinders were removed from the molds, and the ends of the cylinders containing UHPC were ground to level the surface and prepare for testing. These specimens were then placed into the cure room until testing. After curing, these specimens were then tested in compression according to ASTM C39 (per ASTM C882), as shown in Figure 30. The ultimate bond shear stress was then calculated by dividing the recorded maximum load by the area of the bond surface.

a) Substrate half cylinder with surface prepped b) Prepped substrate in cylinders

Figure 29: Typical slant-shear specimen preparation prior to UHPC placement

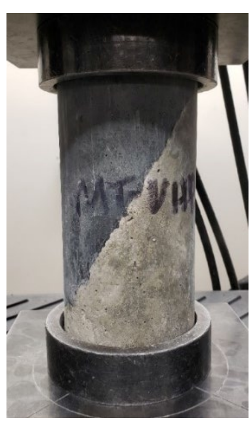


Figure 30: Slant-shear specimen in load frame

3.3 Test Results

3.3.1 Compressive and Tensile Strengths

The average compressive and tensile strengths of the various UHPC at 7, 14, and 28 days are provided Table 5, along with the measured and predicted flexural strengths at 28 days. The compressive and tensile averages were calculated from the results of 3-5 cylinders and 2-3 prisms, respectively. Included in this table are the dynamic and static flows recorded for each UHPC mix. As expected, compressive strength increased with time for all UHPC mixes. The MT-UHPC mix and the Ductal-T mix both reached 28-day compressive and tensile strengths of around 17 ksi and 3.4 ksi, respectively. The MT-UHPC-T mix was observed to have the lowest compressive and tensile strengths (15.4 ksi and 2.8 ksi); however, these strengths are still in line with those expected for UHPC. As previously mentioned, this was the first large-scale batch of a thixotropic MT-UHPC, and further research may be warranted to optimize the admixture dosages, which could have a positive effect on strength.

Regarding ultimate tensile strengths, the strengths are on par with past research on this material. For reference, this table also includes estimates of the tensile strength based on the compressive strength of the material. Specifically, the tensile strengths were predicted as $f_r = 7.5\sqrt{f'_c}$ with f_r and f'_c in psi. As can be observed in this table, the measured tensile strengths are at least three times the predicted values. However, it should be noted that the tensile stress calculated at ultimate load is for comparative purposes, as the equation used to calculate this stress from applied load assumes no cracking and linear-elastic behavior, which is not the case at ultimate load.

Table 5: Average compression and flexure test results

	Flow (in)		Compressive Strength, f'c (ksi)			Ultimate Tensile Strength (ksi)		
UHPC Type	**Static**	**Dynamic**	**7-Day**	**14-Day**	**28-Day**	**Measured**	**Predicted**	**Meas/Pred**
MT-UHPC	10.25	-	14.3	15.1	17	3.37	0.978	3.45
MT-UHPC-T	4	5.5	11.6	-	15.4	2.8	0.931	3.01
Ductal-T	4	6.5	15.1	17.3	17.4	3.43	0.989	3.47

3.3.2 Direct Tension Results

The average compressive strengths on the day of testing are provided in Table 6 for the substrate concrete and the UHPC. It should be noted that the MT-UHPC-T specimens were tested 7 days after casting the UHPC and the specimens for the other two mixes were tested 14 days after casting the UHPC. The results from the direct tension tests are provided in

Table 7, including the averages and coefficients of variation (CoV) observed for each surface preparation method. Each specimen failed at either the bond between the two materials (Figure 31) or in the substrate concrete (Figure 32). The asterisks in the table indicate what type of failure was observed for each specimen. It should be noted that if the specimen failed in the substrate concrete prior to bond failure the actual ultimate tensile bond strength is unknown, and therefore the value provided in the table can be interpreted as a minimum value. It should be noted that some of the core specimens extracted from the slabs were not viable for testing due to incidental damage or poor consolidation, hence the varied number of specimens.

Table 6: Average concrete strengths for direct tension testing

UHPC Type	Substrate Compression (ksi)	UHPC Compression (ksi)
MT-UHPC	5.4	15.1
MT-UHPC-T	5.2	11.6
Ductal-T	5.4	17.3

Table 7: Direct tension results for all specimens

Groove Pattern	Sample Number	MT-UHPC (psi)	MT-UHPC-T (psi)	Ductal-T (psi)	
				Wet	Dry
Typical	T1	280**	239*	197*	60*
	T2	210**	146*	332*	11*
	T3	256**	291*	433*	15*
	T4	251*	192*	367**	106*
	T5	206**	208*	-	-
	T6	234*	-	-	-
	Average	239	215	333	48
	CoV	10.90%	22.60%	25.90%	81.20%
Crosshatch	XH1	220*	148*	343*	-
	XH2	234*	161*	297*	-
	Average	227	155	320	
	CoV	3.20%	4.20%	7.10%	-
Chipped	C1	252**	-	234**	-

*Bond Failure

**Substrate Concrete Failure

Figure 31: Example direct tension failure at the bond (Ductal-T Wet T1)

Figure 32: Example direct tension failure in the substrate concrete (MT-UHPC T2)

As can be seen in Table 7, the average bond strength limits for all specimens ranged from 155 to 333 psi regardless of the surface preparation method (sans the dry substrate preparation), which is above the ACI recommendations of 150 psi for concrete repair [26].

To facilitate a comparison between the different UHPC mixes, the average tensile stresses for the typical (T) specimens are shown in Figure 33 for each UHPC mix type. As can be observed in this figure, both MT-UHPC mixes had similar tensile strengths with the conventional MT-UHPC slightly outperforming the thixotropic mix. The Ductal-T performed the best, with strengths approximately 40% higher than those observed for the other two mixes.

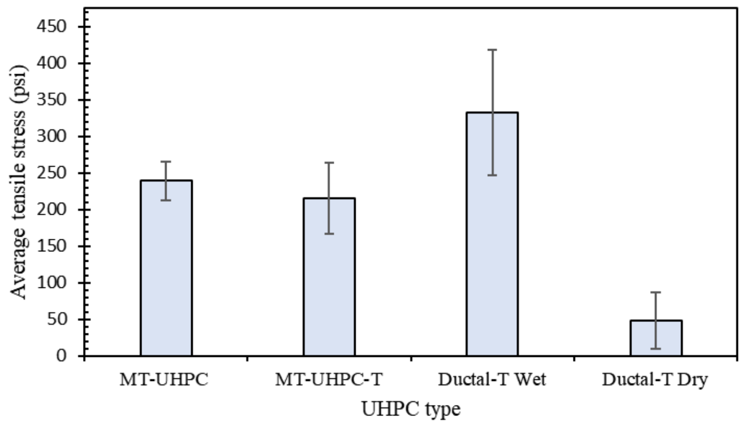


Figure 33: Average peak tensile stresses of typical (T) direct tension specimens (error bars represent one standard deviation).

Regarding the effects of surface preparation methods, the results for the dry Ductal-T specimens highlight the importance of wetting the surface of the substrate concrete prior to UHPC placement, as the average bond strengths observed for these specimens was only 48 psi. Further, for MT-UHPC and Ductal-T, the bond strengths observed for the XH specimens were slightly less than those observed for the T specimens, indicating that this surface preparation does not improve the bond between the layers. The results of the XH specimen for MT-UHPC-T were significantly less than the T specimens, most likely due to the poor consolidation and further highlighting the need to fine-tune the admixture dosage. Similarly, the effect of "chipping" the concrete was shown to have mixed results (increases capacity for one type of concrete, while reducing it for the other).

3.3.3 Slant Shear Results

The slant shear specimens for all UHPC mixes were tested 7 days after casting the UHPC. The average compressive strengths on the day of testing are provided in Table 8, while the measured minimum bond shear strengths are provided in Table 9. Note that all specimens were observed to fail in compression in the substrate concrete (Figure 34), sans one specimen that failed at the interface between the substrate concrete and Ductal-T (Figure 35). Because nearly all specimens failed in the substrate concrete prior to the bond failing, the actual bond shear stress was not obtained, and the values reported here can be interpreted as the minimum bond shear stress. It should be noted that all minimum bond shear stresses were nearly 3 ksi, which far exceeds the ACI specified minimum of 1 ksi [26]. This, despite the surface preparations being parallel to the loading direction, a conservative alignment. To obtain the actual bond stress, future testing could consider wrapping the substrate concrete with fiber reinforced polymer to force the failure to the bond surface.

Table 8: Average concrete strengths for slant shear testing

UHPC Type	Substrate Compression (ksi)	UHPC Compression (ksi)
MT-UHPC	5.4	14.3
MT-UHPC-T	5.2	11.6
Ductal-T	5.6	15.1

Table 9: Slant shear results for all specimens

Sample Number	Minimum Bond Shear Strength (ksi)		
	MT-UHPC	MT-UHPC-T	Ductal-T
1	2.94	3.15	3.13*
2	2.77	3.33	3.26
3	2.75	3.31	3.3
4	2.82	3.37	3.16
Average	2.82	3.29	3.24
CoV	3.02%	2.94%	2.23%

*Bond Failure

a) MT-UHPC-T 1 b) MT-UHPC 2

Figure 34: Example slant shear failures in the substrate concrete

Figure 35: One specimen with a slant shear failure at the bond (Ductal-T 1)

3.4 Summary of Material Level Evaluation

Three UHPC mixes were investigated, including 1) MT-UHPC, 2) MT-UHPC with the addition of a viscosity changing admixture for thixotropy, and 3) a proprietary thixotropic version of Ductal. The workability, and compressive and tensile strengths were evaluated first, followed by direct tension and slant shear bond tests with varying surface preparation methods.

Overall, all three UHPC mixes exhibited similar and adequate compressive and ultimate tensile strengths. Also, all mixes and surface preparation methods/geometries reached the minimum tensile and shear bond strengths recommended by ACI for concrete repair. The only specimens that did not meet the tensile bond minimum were the specimens in which the surface of the substrate concrete was not wetted prior to placement of the UHPC overlay (Figure 33), highlighting the importance of this step. The actual shear bond stresses at failure were not obtained, because all slant shear failures occurred in the substrate concrete, and therefore, the test values can be interpreted as minimum values (that still meet minimum requirements).

Additionally, both thixotropic mixes exhibited appropriate flows; however, the Ductal-T mix proved to be more fine-tuned than the MT-UHPC-T mix with regard to the desired thixotropic properties. The Ductal-T mix had preferable workability and overall better strengths. Due to the additional refinement needed for the MT-UHPC-T mix to match these properties, the Ductal-T mix was chosen for the larger-scale structural testing discussed later in Chapters 5 and 6.

4 UHPC Overlay Projects, Material Specifications, and Implementation Issues

This chapter is focused on what other states have done to implement the use of UHPC for bridge deck overlays. Specifically, the focus is on reviewing existing UHPC overlay projects and investigating what other states have learned about using and specifying UHPC for bridge deck overlays.

4.1 Summary of Existing UHPC Overlay Projects and FHWA Reporting

4.1.1 Existing UHPC Bridge Deck Overlay Projects

Proprietary UHPC has been used in bridge-deck overlays by several states. A summary of selected bridge-deck overlay projects is provided in Table 10. In addition to the projects summarized in Table 10, FHWA reported at least 11 other known projects that have been completed across New Jersey, New York, Illinois, Rhode Island, Indiana, Pennsylvania, and Delaware; however, these projects had minimal published information and have therefore been excluded from the table.

In summary, there were several overarching themes and takeaways from these existing UHPC bridge deck overlay projects. These include the importance of properly performing flow tests to ensure desired material consistency, getting the existing deck to saturated surface-dry (SSD) before casting the overlay, and tarping and/or applying a curing compound soon after placement. Additionally, most projects preferred to install UHPC overlays where the UHPC will be the final riding surface and usually the final surface is diamond ground. When added strength is not a concern, thinner overlays are used to minimize material costs. Thicker UHPC overlays are a good option when bridges need major deck rehabilitation or replacement.

Table 10: Summary of Selected UHPC Overlay Projects [23,25,33]

Project	State	Company	Thickness	Dimension(s)	Volume	Timeline Details
Delaware Memorial Bridge	DE-NJ	UHPC Solutions	Up to 4”	1000’ long	328 yd^3	90 yd^3 at 26’ wide sections in 8-hour shifts
• Used thin lift UHPC paving equipment by GOMACO Corp.						
Bruckner Expressway	NY	UHPC Solutions	2”	--	14 yd^3	9 days and two stages
• Bridge had damaged T beams and UHPC was used to add strength. • Surface was already prepared. • Used crane and concrete bucket to deliver material.						
NJ 159 WB over Passaic River	NJ	UHPC Solutions	2.75”	--	65 yd^3	36 yd^3 in 6-hour stages
• Used Thin Lift UHPC Paver and used two high shear mixers and buggies to deliver. • Removed asphalt and scarified 0.5” of existing concrete deck with Hydrodemolition.						
I-280 WB over Newark Turnpike	NJ	UHPC Solutions	1.5”	340’ long	124 yd^3	Two stages
• Also included a non-thixotropic mix for expansion joint headers. It is not clear if the stated volume includes UHPC for headers. • Overlay was then covered in 2.25” of asphalt per common European practice.						
NJ 57 over Hances Brook	NJ	UHPC Solutions	1.5”	25’ long	6 yd^3	Two stages
• Used stepped longitudinal construction joint with galvanized rebar between phases. • Used a vibratory screed. • Overlayed final with 2.25” of asphalt.						
SR-1 Little Heaven – 2 bridges	DE	UHPC Solutions	1.75”-5” average 3”	120’ long, 42’ wide	96 yd^3	3 casting days in Feb. Over 2 weeks.
• 0.25” Hydrodemolition surface preparation. • Heating accomplished with forced air hydronic systems above and below bridge deck. Also heated UHPC premix and mixing water and tented and heated afterwards. • Varying depths were because of unevenly cambered steel beams. • Reached design strengths of 11-13 ksi in 3 days.						
Bridge over Floyd River	IA	UHPC Solutions Cramer & Associates Inc. Walo Iowa LLC as a sub	1.75”	205’ long, 44’ wide	*48.7 yd^3	Machine placed. Less than 10 workdays. One lane remained open.
• Used a UHPC waffle deck configuration with thin UHPC overlay.						
Mud Creek Bridge	IA	Iowa DOT	1.5”	100’ long, 28’ wide	*13 yd^3	2 separate days with 3 days in between
• Used a regular vibratory screed.						
Bridge No. 7032 in New Mexico	NM	NM DOT	1”	300’ long, 51’-54’ wide	*81 yd^3	105 batches, 0.77 yd^3 each
• Used a non-proprietary thixotropic UHPC mix. • 19.5 ksi. design 28-day compressive strength. Only reached 13.6 ksi – attributed to lack of precise water content control and differences in initial curing temperatures. • Performed direct tension testing on 9 specimens, with 239 psi average. This included 5 epoxy failures, so actual bond strength is larger.						

*Volumes estimated based on available data.

4.1.2 FHWA Documentation

FHWA [34, 35] summarized the results of previous overlay and repair projects, and developed recommendations for the successful implementation in these applications. Their report first provides overall material specifications for UHPC, and then discusses design and construction specific considerations. The material specifications for the UHPC in these applications are summarized in Table 11.

Table 11: Summary of UHPC Material Properties [34]

Property	Variable Symbol	Acceptance Criteria	Test Method
Compressive Strength	f'_c	18 ksi	ASTM C39 and ASTM C1856
Effective Cracking Strength	$f_{t,cr}$	0.75 ksi	AASHTO T397
Localization Stress	$f_{t,loc}$	$f_{t,loc} \geq f_{t,cr}$	AASHTO T397
Localization strain in direct tension		0.0025	AASHTO T397
Steel fiber reinforcement	V_f	2% by volume – 3.25% for overlays	NA
Rheology/Workability	N/A	Varies by supplier. Typ. hold profile at slope of 10% Ex - Flows of 6 to 8 inches for slopes of 6%	Modification of ASTM C1856 - Dynamic flow table test (20 drops)
Unit Weight	N/A	155 lb/ft^3 (for 2% fibers)	N/A
Chloride Ion Diffusion Coefficient	N/A	2 E-10 in^2/s	N/A
Coefficient of thermal expansion	N/A	7 E-6 inches/inch/°F	N/A
Modulus of elasticity	E_c	2500 $(f'_c)^{1/3}$	N/A
Bond strength to existing concrete	NA	0.35-0.6 ksi	N/A

As discussed above, this document also provides design and construction specific recommendations. For example, it provides recommendations for development length, lap splice length, minimum cover and spacing of reinforcing bars, formwork and traffic vibration mitigation, mixing methods, placement and consolidation, curing, and strength gain. Additionally, recommendations specific to UHPC overlays are discussed, including material consistency, fiber content, thickness, clear spacing, and cover, existing deck concrete substrate preparation, skid resistance, phased construction joints, existing deck surface preparation, placing and finishing equipment and methods, and postconstruction concerns. A complete list of recommendations with pertinent details is included in Appendix B of this report.

4.2 State UHPC Overlay Material Specifications and/or Special Provisions

While many states have used UHPC in construction applications, only four have developed specifications or special provisions specifically for UHPC overlays (Iowa, New Jersey, New Mexico, New York). This section briefly discusses how the specifications/provisions from these four states vary from or supplement the FHWA recommendations.

4.2.1 Iowa DOT Special Provisions for UHPC Overlay

One of the primary differences between Iowa [36] and FHWA [34] is a lower required 28-day compressive strength of 14 ksi, compared to 18 ksi from FHWA. Also, their provision specifies compressive testing according to AASHTO T22 instead of ASTM C39/C1856 specified by FHWA. Iowa also includes supplemental material properties, summarized in Table 12.

Table 12: Summary of Material Properties (supplemental to FHWA) included in Iowa UHCP Specification [36]

Property	Acceptance Criteria	Test Method	Frequency
Compressive strength	14 ksi	AASHTO T22	12 tests in 1st day at intervals specified by engineer, 2-day, 3-day, 4-day, 8-day, 14-day, & 28-day
Long term shrinkage	≤ 800 Micro-strain (64 weeks)	AASHTO T160	
Chloride ion penetrability	< 0.1183 lbs/yd^3 (0.5" depth)	AASHTO T256	
Rapid chloride ion penetrability	≤ 350 coulombs	AASHTO T277/ ASTM C1202	2 per job (during field placement)
Scaling resistance	Y < 3	ASTM C672	
Freeze-thaw resistance	Relative dynamic modulus of elasticity > 95% (300 cycles)	AASHTO T161 and ASTM C666A	
Alkali-silica reaction	Innocuous	ASTM C1260	
Slump flow and visual stability	7 inches to 10 inches, no bleed water, consistent fiber distribution	ASTM C1437/ASTM C1611	1 per batch

Compared to FHWA, Iowa provides more specifics on the constituent materials used in the UHPC mix. Specifically, they discuss the requirements for the fine aggregate, cementitious material, steel fibers, water, and admixtures. Also, the provisions specify that the fine aggregates and cementitious materials must be premixed, proportioned in bags/supersacks, and come from the same batch or lot.

Iowa also provides details on including a placement plan with a detailed construction work schedule, which must be reviewed by the engineer and serves as a guide for the contractor to reference. Specific details on what should be included in the placement plan are listed in Appendix C. A preconstruction meeting between representatives of the UHPC manufacturer, contractor, and other interested parties is required to approve the placement plan and no UHPC placement is permitted before this meeting occurs.

Some other notable differences and/or supplemental details that Iowa provides, compared to FHWA, dealing with construction considerations include the following bullets:

- Two UHPC manufacturer representatives are required on site at all times.
- Pumping UHPC is not allowed.
- UHPC must be kept from freezing until a minimum of 11 ksi compressive strength is reached.
- A minimum of three portable batching units are required.
- Finished surface preparation is not allowed until a minimum of 11 ksi compressive strength is reached and a minimum of 3 curing days has occurred.

- The method of UHPC measurement is in square yards of placed and accepted material. Volume is computed using plan dimensions and the grinding quantity is not measured.
- Payment is based on unit price per square yard. Pricing includes surface preparation, supplying, mixing, transporting, placing, finishing, curing, grinding, grooving, and furnishing all equipment tools, labor, and incidentals required.

4.2.2 New Jersey Performance Specification Section 515 – UHPC Overlay

New Jersey [37] follows many of the same requirements as Iowa [36]. Like Iowa, a placement plan is required, following similar guidelines; however, the New Jersey placement plan also includes sections for quality control of mixing time and batch times, and for cold weather placement procedures, when appropriate. Additionally, similar acceptance criteria to Iowa are followed, but also include the tension criteria listed in Table 13.

Table 13: Summary of Supplemental Material Properties Listed in New Jersey UHPC Specifications [37]

Property	Acceptance Criteria	Test Method
Direct tension cracking strength	≥ 1,100 psi	FHWA-HRT-17-053
Direct tension sustained post-cracking tensile strength	≥ 1,250 psi	FHWA-HRT-17-053
Direct Tension Bond Strength	100% failure in substrate concrete with concrete compressive strength ≥ 4 ksi	ASTM C1583, bonded to exposed aggregate concrete surface
Modulus of Elasticity	≥ 6,500 ksi	AASHTO T256

Some other notable differences and/or supplemental details that New Jersey provides, compared to FHWA and Iowa, dealing with construction considerations include the following bullets:

- Pumping is allowed if it is successfully demonstrated at least 30 days prior to placement.
- Construction joints must be provided at stage lines (including galvanized reinforcement steel), and additional joints are only allowed with prior approval. Additional joints not already approved will not be the basis for additional payment or a time extension.
- Rapid chloride ion penetrability maximum is limited to 250 coulombs.
- The finished overlay surface profile must match the proposed within ±1/4 inch.
- When the UHPC overlay is the final riding surface, a temporary surface above the final grade must be included to facilitate room for diamond grinding.
- At least 60 days prior to the proposed placement, a 4’ x 12’ x 3” rectangular slab must be cast at an 8% grade. Six cores must be taken and have depths within ½” of 3”.
- Because New Jersey also has a performance-based specification, there is a section on qualification testing of 12 cylinders, 3” x 6”, for compression following ASTM C39.

4.2.3 New Mexico Special Provisions for Section 512-B: UHPC Overlay

New Mexico [38] also follows most of the same requirements as Iowa [36] and New Jersey [37], but also provides the supplemental material acceptance criteria summarized in Table 14.

Table 14: Summary of Supplemental Material Properties Listed in New Mexico UHPC Specifications [38]

Property	Acceptance Criteria	Test Method
Flexural strength	Pp/P1 > 1.4	ASTM C1609 with C1856/1856M modifications
Abrasion Resistance	< 0.1 ounces lost	ASTM C944 with C1856 modifications, double load abrasion device, 6" cores
Water/Binder ratio	≤ 0.28	

In addition to some other minor differences, New Mexico specifically specifies using the Ductal product line and the minimum 28-day compressive strength is increased to 18 ksi (matching FHWA [34] recommendations). Some other notable differences and/or supplemental details that New Mexico provides, compared to FHWA, Iowa, and New Jersey, dealing with construction considerations include the following bullets:

- High molecular weight methacrylate (HMWM) should be applied to any unacceptable cracks.
- Multiple meetings are required prior to the placement date to approve similar details to the placement plan discussed above for Iowa, including a mock-up pour.
- The minimum surface roughness is specified at an average height of 0.25".
- Placement requires special approval if the relative humidity drops below 35%.
- Employing the maturity method to determine in-situ strength is allowed using the strength-maturity relationship recommended by the manufacturer. The relationship must be regularly validated and any changes in mix design require a new strength-maturity to be developed.
- The product representatives present at the pours are required to have experience spanning at least 3 years or 5 projects.

4.2.4 New York Performance Specification Item 578.21010001 – UHPC Overlay

New York [39] follows most of the same requirements as Iowa [36], New Jersey [37], and New Mexico [38]. New York also follows the same 28-day compression strength requirements as New Mexico (and FHWA), with a value of 18 ksi. In addition to some minor material property adjustments, New York also specifies a 24-hour compression strength of 12 ksi and a prism flexural tensile toughness of $I_{30} \geq 48$. The other notable additions that New York provides, compared to FHWA and the other states, dealing with construction considerations include the following bullets:

- The same 4' x 12' x 3" test slab as New Jersey is required for coring.
- A pre-pour meeting is required, but an official placement plan is not mentioned.
- For payment, quantities shall be measured to the nearest cubic foot rather than square yard.

4.3 Summary of UHPC Overlay Projects, Material Specifications, and Implementation Issues

The recent FHWA report on UHPC-based preservation and repair methods served as the primary source for this work. Recommendations from this report are compared to those from UHPC-related material specifications/provisions from four state DOTs. Overall, the success that other states have had in using UHPC for overlays is very promising for its potential use in an overlay implementation project in Montana.

5 Structural Testing Experimental Design

This chapter details the experimental design for the structural testing completed in this research, which was focused on quantifying the effects that UHPC overlays have on the behavior and capacity of existing bridge decks. First, for a brief introduction, previous UHPC overlay deck testing that has been performed by other researchers is presented. The experimental design for the current research and the construction of the test specimens is then discussed. Overall, five slab specimens were tested in flexure via 3-Point bending, and the slab dimensions/detailing were designed to compare results for differing overlay depths, substrate concrete strengths, and positive vs. negative bending behaviors.

5.1 Previous UHPC Overlay Strength Testing

A review of previous research on slabs with UHPC overlays was included in Chapter 2 and Table 15 provides a summary of the key aspects. The goal of this review was to determine which aspects of UHPC overlay structural design had been experimentally tested and what additional information would be ideal to gather as part of the current study. The key takeaways from the literature search were that only Iowa State University has conducted strength testing on slabs with UHPC overlays and all failures observed were shear failures initiated in the substrate concrete. The test slabs had a minimum depth of 8” (not including the overlay). Although current bridge deck design in Montana does not allow for a deck depth less than 8”, there are many older bridges with thinner decks, as they were (of course) designed based on old requirements. These bridges may be the best candidates for UHPC overlays in the future, and for this reason, the structural testing of the current project is focused on thinner decks than what have been tested by others.

Table 15: Summary of previous UHPC structural slab testing [23, 24, 40]

Org.	Testing Performed	Overlay Thickness	Results	Notes
Iowa State University	Overlay on fabricated ‘precast’ element. 2’x8’x8” with standard AASHTO reinforcement. Tested on 6 ft clear span, with steel plate (10”x20”) in the middle for truck tire. First loaded to just above cracking values. Then loaded to cracking and failure.	1.25”	All eventually failed in shear in the NC at 70 kips. 4.4 times the design load. No slip observed. Estimated shear capacities were much lower than slant shear, due to substrate concrete failing. UHPC could experience much higher. They then created a model to estimate moment curvature that was within 10% of measured values.	Used non-thixotropic mix. Tarped cured for 2 days then air cured. Some had delamination from drying shrinkage.
Iowa State University	Continuation of above, but tested no overlay, overlay on top with reinforcement and on bottom with reinforcement.	1.5”	Observed an increase in strength, though wire mesh didn’t contribute much to negative moment because the area was fairly small. More steel could help, though would increase weight.	Maybe if we tested a slab that was as thick as the ones with the overlay to better compare. Baseline without overlay also good to test.
New Mexico State University	Slab tests for shrinkage, channel girder with and without overlay. Cyclic loading, 4-point flexural testing to a set midspan deflection. 1000 load-unload cycles. 2 cycles per min for first 100. Then 4 cycles per min for next 900. Then loaded to failure.	1”	Steel helped with shrinkage. 27% increase in strength for same 0.4” deflection. Maintained elastic behavior.	Measured roughness with sand test ASTM E965. Had a 1000-kip capacity compression machine.

5.2 Experimental Design

5.2.1 Overview

Five slab designs were tested in this research. The key design parameters (span, thickness, rebar reinforcement, etc.) are based on the existing Fred Robinson Bridge, which spans the Missouri River on Highway 191 in Northeastern Montana. A cross-section of the existing bridge deck is shown in Figure 36.

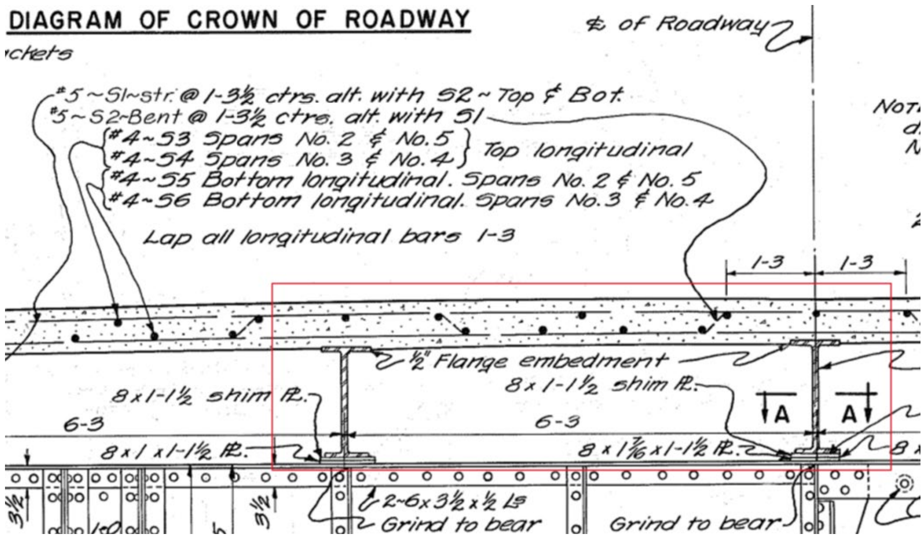


Figure 36: Section from Fred Robinson Bridge drawings. The red box shows the approximate location of the test slabs' representation of the bridge deck and the girder supports shown later in Figure 37.

The test specimens in this research represent small sections of the overall bridge deck; specifically, four of the test specimens are intended to replicate the slab sections in positive bending spanning between the girders and one of the test specimens is intended to represent the slab section in negative bending spanning over the girders. An isometric view of a typical slab specimen is shown in Figure 37. Note that more specific details of the various specimens will be covered in a later section.

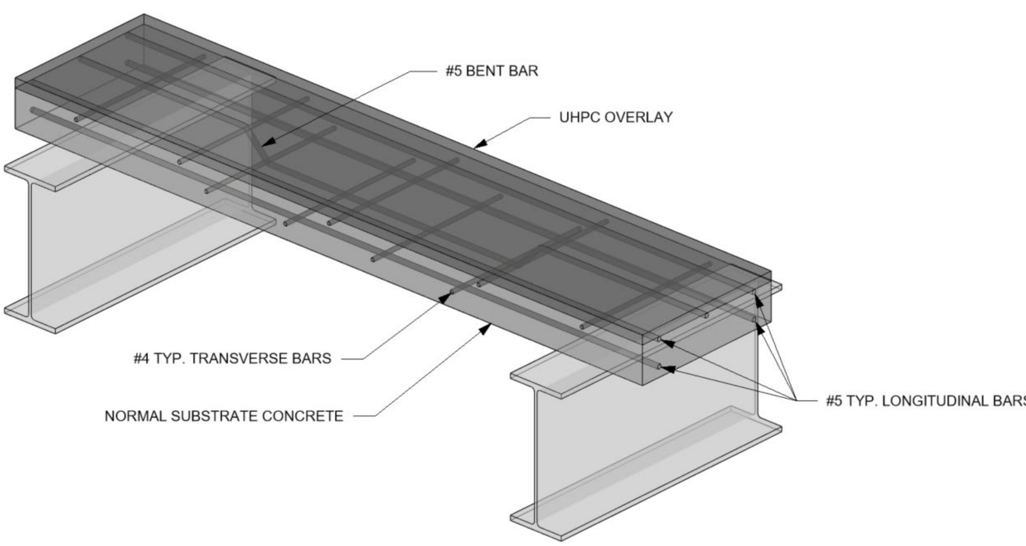


Figure 37: Test Slab Isometric View (with an example UHPC overlay shown).

5.2.2 Loading, Supports, and Data Acquisition

The slab specimens were tested in 3-Point bending, similar to the research conducted at Iowa State University, with the load applied following AASHTO recommendations for standard tire load and orientation. In this setup (Figure 38 and Figure 39), the specimens were simply supported, and a load was applied at midspan with a hydraulic actuator. The load was transferred to the specimen with a 10" by 20" steel plate, intended to represent a standard tire load between the girders. The specimens were tested until failure while recording the applied load and resultant displacements. The load was recorded with a load cell attached to the end of the actuator, and string potentiometers were used to record the resultant displacements. The displacements were recorded on both sides of the specimens at the midspan and on one side of the specimen at the quarter spans. A GoPro was used to record videos during all tests. All slabs were loaded to a midspan displacement of at least 3.5" (4.7% of span length).

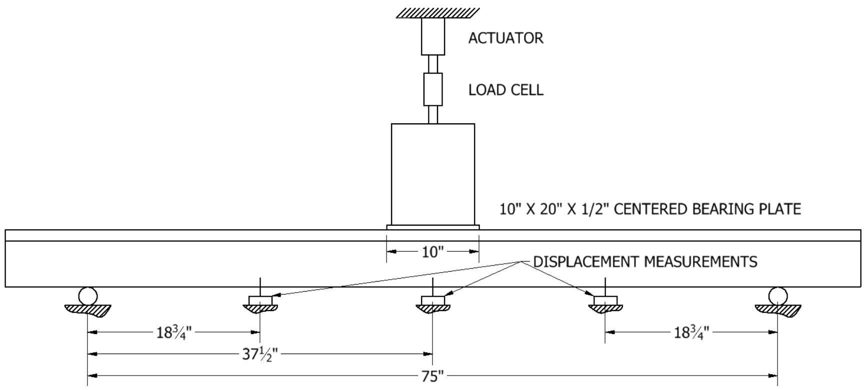
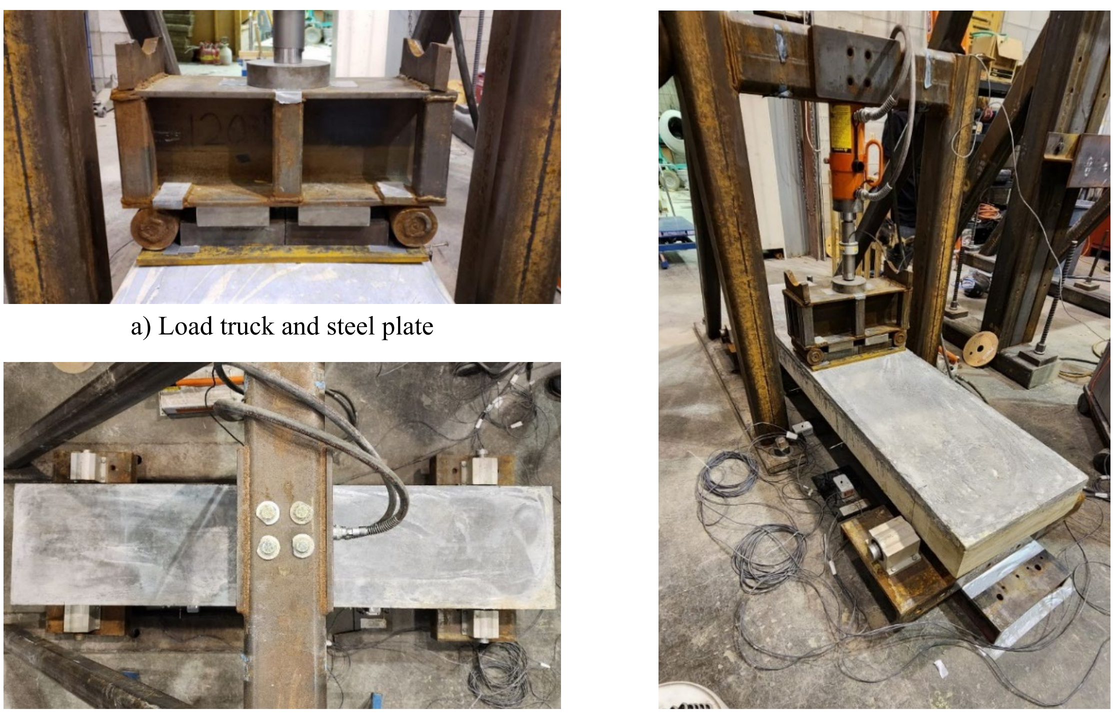


Figure 38: Schematic of test set-up.



a) Load truck and steel plate

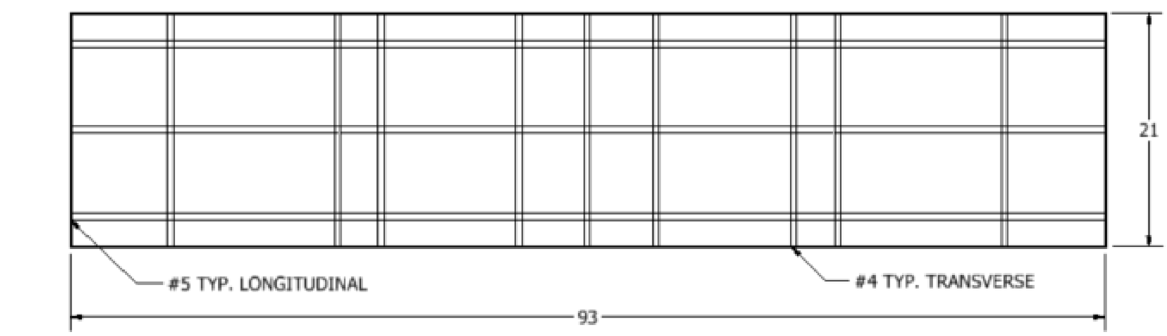
b) Top view of slab

c) Isometric view of entire setup

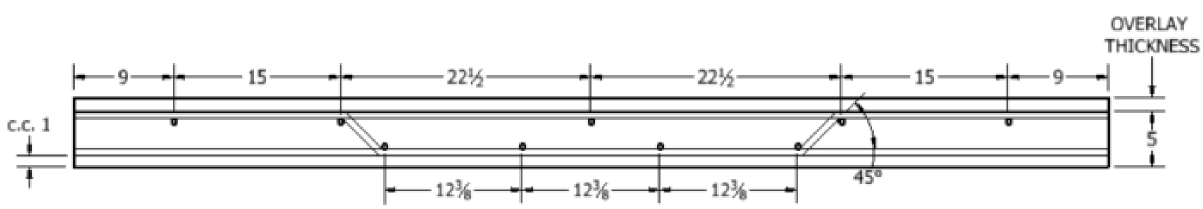
Figure 39: Test set-up

5.2.3 Test Slab Details

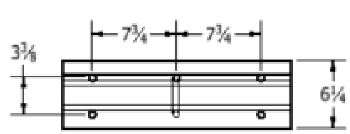
As discussed above, the slab specimens are intended to represent sections of a bridge deck slab spanning between the longitudinal bridge girders (and one that represents a slab spanning over the girders subjected to negative moment). The primary slab dimensions and reinforcement details mimic those found in the existing Fred Robinson bridge. All specimens had the same width (21 inches), length (93 inches), test span (75 inches), and reinforcement details, but varied in the depth of UHPC overlay, loading condition, and concrete strength. The reinforcement details for all specimens (Figure 40) include longitudinal steel consisting of an alternating pattern of #5 grade 60 rebar. There is one set with a top and bottom bar (outside edges of the test slabs) and another with a bent that drops down between the supports (refer to bent bar label in Figure 37 isometric view). The transverse steel consists of #4 bars spread across both the top and between the bent longitudinal #5 on the bottom. The top clear cover is dependent on the overlay thickness and bottom clear cover is 1”. The longitudinal rebar for one test slab is in a flipped orientation and will be discussed below.



a) Top View



b) Front View



c) Side View

Figure 40: Test slab dimensions and reinforcement details (all dimensions shown in inches).

The 5 test specimens consisted of one control slab, two slabs with varying UHPC overlay depths, one with UHPC overlay with weak substrate concrete, and one tested in negative bending (with UHPC overlay on the bottom in test setup). The specifics of the 5 test slabs are presented below. The naming convention chosen for the slabs is Control for the control slab and then follows a “O – P/N R/W” naming convention for the overlay slabs, where the first letter signifies **O**verlay, the second letter **P**ositive or **N**egative for the moment loading, and the third letter **R**egular or **W**eak for the substrate concrete strength.

1. **Control**: a 6.25-inch thick slab made with regular-strength concrete (~4 ksi).
2. **O-PR1**: *Overlay-Positive Regular 1* – a slab specimen with target overlay thickness of 1.5 inches to be tested in positive moment with regular substrate concrete (~4 ksi).
3. **O-PR2**: *Overlay-Positive Regular 2* – a slab specimen with target overlay thickness slightly more than the 1.5 inches specified in O-PR1 to be tested in positive moment with regular substrate concrete (~4 ksi).
4. **O-PW**: *Overlay-Positive Weak* – a slab specimen with target overlay thickness slightly more than the 1.5 inches to be tested in positive moment with weak substrate concrete (~2 ksi).
5. **O-NR**: *Overlay-Negative Regular* – a slab specimen with target overlay thickness slightly more than the 1.5 inches to be tested in negative moment with regular substrate concrete (~4 ksi). Note that this specimen was tested using the same test setup as the other specimens, but the UHPC was on the bottom of the slab during testing, and therefore in tension. Additionally, the reinforcement was flipped before casting (i.e., the center bent bar is flipped in Figure 37), so that the areas of "top" and "bottom" steel are consistent with the other slabs.

Overall, the control slab will yield a baseline for which to compare the results of the other slabs and the test results from the O-PR1 and O-PR2 slabs will yield a comparison of slightly differing overlay thicknesses. Deck concrete cores taken from older bridges can sometimes yield lower than expected strengths. The test results from the O-PW slab will shed light on how a UHPC overlay will affect the capacity on a deck with weak substrate concrete. Bridge decks are subjected to loading that causes both positive and negative moments. In the negative moment regions, the overlay will be in tension and the UHPC will increase the tensile capacity of the slab. The test results from the O-NR slab will help better understand this behavior.

5.3 Specimen Construction and Preparation

5.3.1 Formwork and Reinforcement

Figure 41 shows the formwork and reinforcement for a typical specimen during construction. The formwork was constructed using 0.75-inch plywood sheets and 2-by-8-inch fur boards. The bottom rebar was placed on 1-inch chairs at the bottom of the forms, while the top reinforcement was hung from the top of the forms, as shown. Form oil was applied to the forms prior to casting.

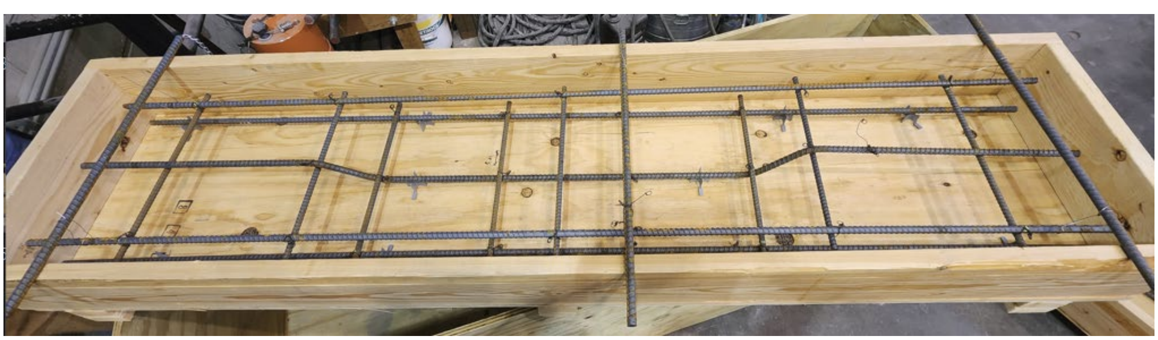


Figure 41: Example slab formwork and reinforcement before casting.

5.3.2 Substrate Concrete Surface Preparation

The surface of an existing slab will need to be prepared for a UHPC overlay. This process can be done several ways; however, MDT and the FHWA recommend hydrodemolition. A surface retarder (MasterFinish XR Lilac, Appendix D) was used in this research to simulate a hydrodemolitioned surface. Test panels were cast to test the surface retarder application procedures and allowable set times and to gain confidence in creating the final desired surfaces. Based on the test panel findings, the slab specimen surfaces were heavily coated and sprayed off at around 20 hours. Overall, after experimenting on the test panels (Figure 42), the researchers felt confident that the desired surface preparations would be achieved on the full-scale slabs' substrate concrete. Also, due to the recommendations on the technical data sheet, additional test panels were made when the slabs' substrate concrete was cast to have surfaces to test spray to ensure desired behavior before preparing the slabs themselves.

Figure 42: "Hydrodemolition" Test Slab (Left Sprayed at 7 hours. Right Sprayed at 20 hours.)

5.3.3 Weak Mix Concrete Trials

For the O-PW slab, a weak concrete mix was developed. Due to the atypical nature of designing a 2 ksi strength concrete, a few trial batches were performed to fine tune the mix design. Using a PCA Table 12-3, the water cement ratio of a standard 5 ksi mix from MSU was altered for a weaker mix [40]. Trial batches (0.2 ft^3 volume) were cast with water cement ratios ranging from 0.8 to 0.85 to reduce the strength. After several tests, it was decided to use the mix design shown in Table 16 for a 3-ft^3 batch, as it would yield a mix with ~2 ksi strength at the time the slab would be tested.

Table 16: Weak substrate concrete mix design for a 3 ft^3 batch

Item	Weight (lbs)
Water	44.6
Cement	55.3
Coarse Aggregate	218.4
Fine Aggregate	116.1

Additionally, the substrate concrete surface preparation technique was tested on the weak mix. A test panel was cast and prepped following the process described previously. The surface had adequate results after being sprayed at 24 hours, though it was noted that it took much longer for the surface bleed water to dissipate.

5.3.4 Casting Substrate Concrete

The four regular strength substrate concrete slabs (Control, O-PR1, O-PR2, and O-NR) were cast on the same day using a 4 ksi mix provided by Quality Ready Mix in Bozeman, MT. Each slab was vibrated and filled to the appropriate height (just above the rebar with concrete paste for the overlay specimens and to 6.25" for the control specimen). After all surface water had evaporated, the tops of all slabs except the control were coated with a thick layer of the surface retarder (Figure 43a). The control slab was simply covered in saran wrap. After about 20 hours, the top layers of the overlay specimens were sprayed and brushed off (Figure 43b). The slabs were all covered with saran wrap and left to cure for one additional day before being moved, stacked, covered in wet towels, and wrapped in a thin plastic sheet (Figure 44). It should be noted that the substrate concrete depth varied along the length of individual slabs and between all of the slab specimens. This was likely due to slight differences in the location of the top rebar, and due to variations in the amount of concrete removed from the top surface. These variations in depth led to variations in the UHPC overlay thicknesses, which will be discussed in a later section.

The substrate concrete for the O-PW slab was cast on a separate day using the mix design discussed in the previous section. It should be noted that it took over an hour for the surface water to evaporate due to the high water content of the mix, after which the surface retarder was applied.

a) After surface retarder applied

b) After spray/brush off

Figure 43: Example slab substrate concrete surface preparation.

Figure 44: Formwork stacked for curing.

5.3.5 Casting UHPC Overlays

The UHPC used for this research was a modified version of Ductal provided by Lafarge Holcim. It should be noted that Lafarge Holcim recommended a slightly different mix compared to what was tested during Task 2 that included a small amount of Chryso Premia 150 admixture that could be added to reach the target flows of 5" for static and 7" for dynamic. The mix design for a 3-ft^3 batch is shown in Table 17. For all slabs except O-PW, the UHPC overlay was cast 14 days after the substrate had been cast. Two batches were required to overlay the specimens. The mix procedures followed those described in the Task 2 Report, with the Premia 150 admixture being added in increments after the steel fibers were mixed until the target flows were reached. The resultant static and dynamic flows for these two mixes are provided in Table 18, and examples of the static and dynamic flows are shown in Figure 45.

Table 17: Ductal-T Overlay mix design for a 3 ft^3 batch

Item	Weight (lbs)
Water	31.8
F5 Admixture	4.1
Chryso Premia 150	0.5
Steel Fibers	46.5
Ductal Premix	375.0

The UHPC overlay for the O-PW slab was cast 3 days after its substrate concrete was cast (Mix 3, Table 18), using the same UHPC mix design and procedures used in the previous specimens.

Table 18: Static and dynamic flow values

UHPC Mix (slabs)	Flow (in.)	
	Static	Dynamic
Mix 1 (O-PR2 and ~O-PR1)	6.0	8.0
Mix 2 (O-NR and ~O-PR1)	5.5	7.5
Mix 3 (O-PW)	5.0	7.0

a) Static b) Dynamic

Figure 45: Example UHPC flow tests for Mix 1.

For all UHPC mixes, the forms were aggressively hit with a rubber mallet to consolidate the material. After casting the UHPC and after the surface water evaporated, the slabs were coated with a thick layer of curing compound (WR Meadows 1600 White, recommended by Ductal, Appendix E). The slabs during and after the application of the curing compound are shown in Figure 46a and b. Additionally, the slabs were covered in a layer of plastic wrap to further protect the UHPC (Figure 46c). After 24 hours, the curing compound was sprayed off with a pressure washer and scraped with a wire brush until almost all had been removed (Figure 47). The slabs were then covered with wet towels, stacked, and covered with a plastic sheet in the same setup as before (Figure 44).

a) Application b) Finished surface c) Covered in saran wrap

Figure 46: Applying curing compound to UHPC overlay surface.

a) Spraying off curing compound b) Final finished surface

Figure 47: Final UHPC surface preparation steps.

5.3.6 As-built Dimensions and Material Properties

Key as-built dimensions and material properties are provided in Table 19. Note that the depth measurements of the substrate concrete and UHPC overlay were taken with calipers for all slabs on both sides at center span and the average values are provided in the table. For all slabs, the top steel consisted of three #5 bars and the bottom steel consisted of two #5 bars. Table 19 also includes the distance from the extreme compressive fiber to the top reinforcement, d_t, and the distance from the from the extreme compressive fiber to the bottom steel, d_b. These values along with the concrete compressive strengths are required for the ACI/FHWA comparison calculations discussed in a later section. The substrate and UHPC concretes compressive strengths were tested according to ASTM C39 and ASTM C1856 respectively. The steel rebar was tested in tension according to ASTM A370-22 and the yield strength was equal to 71.1 ksi.

Table 19: As-built Dimensions and Material Properties

Slab	Layer	Dimensions (in.)			Slab Test Day Compressive Strength, f'c (ksi)	
		Midpoint Depth	d_t	d_b	UHPC	Substrate Concrete
Control	Total	6.28	1.59	4.97	18.3	4.1
O-PR1	Substrate	4.96	1.86	5.2		
	Overlay	1.55				
	Total	6.51				
O-PR2	Substrate	4.82	2.14	5.33		
	Overlay	1.83				
	Total	6.64				
O-NR	Substrate	5.13	1.31	4.82		
	Overlay	1.55				
	Total	6.68				
O-PW	Substrate	4.50	1.54	4.42	17.6	2.7
	Overlay	1.23				
	Total	5.73				

6 Structural Testing Results

As discussed in the previous chapter, all specimens were tested to failure in the Structures Lab at MSU. This chapter details the results for the individual slab specimens, along with key takeaways from these tests. First, individual slab test results will be discussed, and then comparisons will be made for differing overlay depths, substrate concrete strengths, and positive vs. negative bending behaviors. Finally, the measured slab capacities will be compared to ACI and FHWA calculations for predicted capacities.

6.1 Control

The force-displacement curve for the control specimen is shown in Figure 48. Overall, this specimen behaved and failed as expected. After applying load, flexure cracks formed at the bottom of the specimen near the center at a load of around 4 kips, marked by the change in slope of the force-deflection curve. After cracking, the slab continued to gain strength as the rebar was engaged up to 16.7 kips, at which point the steel yielded and the load began to level off. The maximum load for this specimen was 21.5 kips, which occurred at a deflection of 1.3 inches. Ultimately, the slab failed due to concrete crushing at the midspan (Figure 49) with around 4 inches of deflection. The slab demonstrated good ductility, carrying 87% of the ultimate capacity until failure.

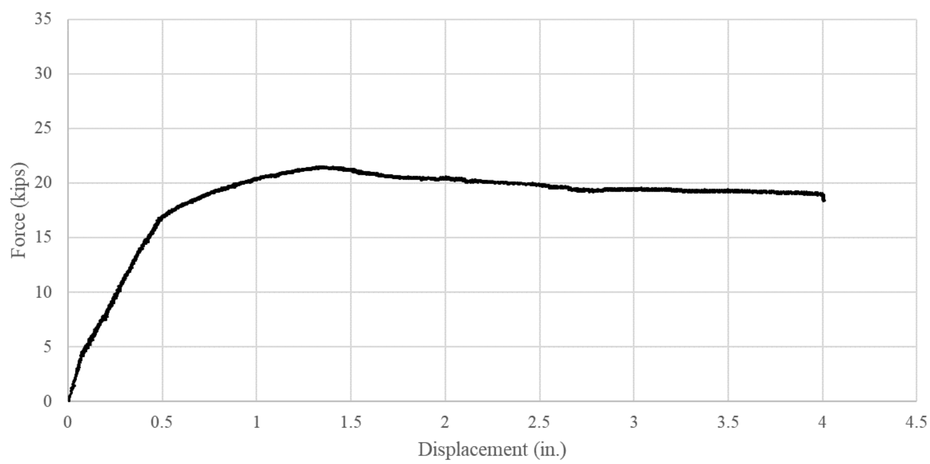


Figure 48: Control slab force-displacement graph.

a) Initial Loading

b) Concrete crushing prior to failing

Figure 49: Control slab during testing.

6.2 O-PR1 (Overlay-Positive Regular, specimen 1)

As mentioned previously, O-PR1 was similar to the control specimen but had a ~1.5 inch UHPC overlay on the top of the specimen. The force-displacement graph for this specimen is shown in Figure 50. After loading, the first sign of distress occurred when the flexural cracks formed at the bottom of the specimen (at approximately 4-5 kips). Again, this is marked by a change in stiffness at this load. The specimen continued to gain strength until the steel yielded at around 20 kips where there is a gradual reduction of stiffness. The specimen then continued to gain strength until it ultimately failed due to the formation of a shear crack in the substrate concrete. This shear crack then propagated to the bond interface between the UHPC and substrate concrete, which then led to the UHPC overlay debonding across the center of the specimen (Figure 51b). The ultimate load occurred at 29.4 kips and at a displacement of around 3.3 inches. It should be noted that there was no distress observed in the UHPC prior to the debonding caused by the shear-crack propagation. It should also be noted that the slab specimen continued to carry 83% of the maximum load even after the sudden drop of load due to the formation of the shear crack. The slab then sustained load after debonding at a capacity close to that of the control slab. After the brittle drop in load, the slab recovered and sustained around 83% of the maximum load until testing was stopped at a displacement of 4.1 inches.

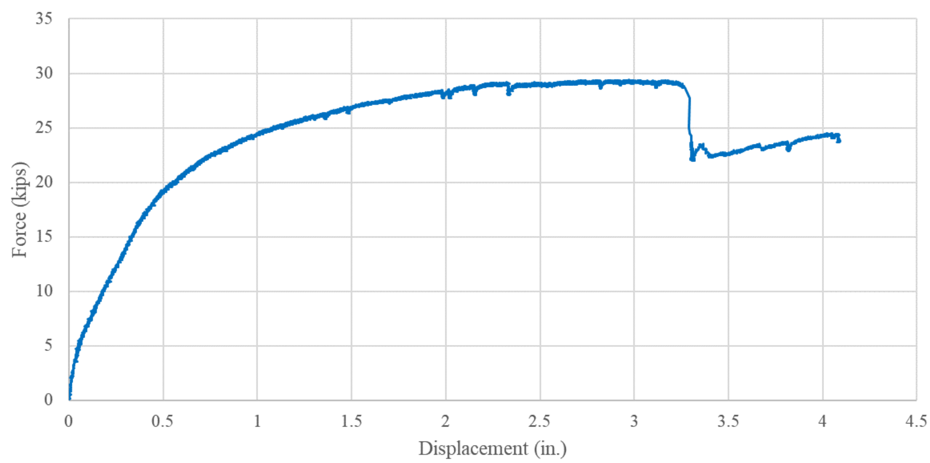
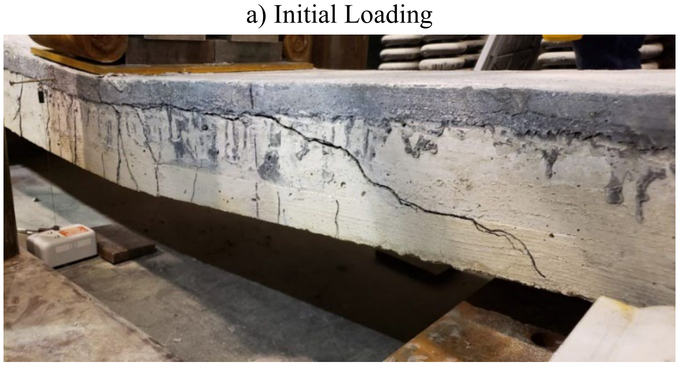
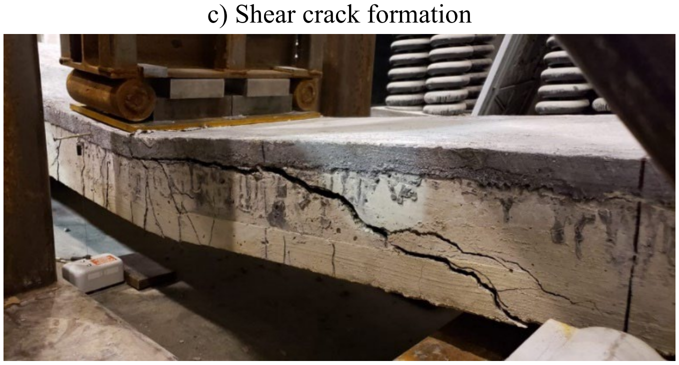


Figure 50: O-PR1 slab force-displacement graph.

a) Initial Loading



c) Shear crack formation



c) Crack widening/spreading at the end of testing

Figure 51: O-PR1 slab during testing.

6.3 O-PR2 (Overlay-Positive Regular, specimen 2)

As mentioned previously, O-PR2 was very similar to O-PR1 but had had a slightly thicker UHPC overlay on the top of the specimen, thus its behavior was very similar. The force-displacement graph for this specimen is shown in Figure 52. After loading, the first sign of distress again occurred when the flexural cracks formed at the bottom of the specimen (at approximately 4-5 kips). Again, this is marked by a change in stiffness at this load. The specimen continued to gain strength until the steel yielded at around 21 kips where there is a gradual reduction of stiffness. The specimen then continued to gain strength until it ultimately failed due to the formation of a shear crack in the substrate concrete. This shear crack propagated to the UHPC overlay, where unlike what was observed in O-PR1, it continued into the UHPC (Figure 53b). The shear crack formed at a load of 32.5 kips and a displacement of around 2.3 inches. It should be noted though, that the slab continued to carry a significant load after the initial formation of the shear crack, and ultimately failed when the shear crack completely propagated through the UHPC.

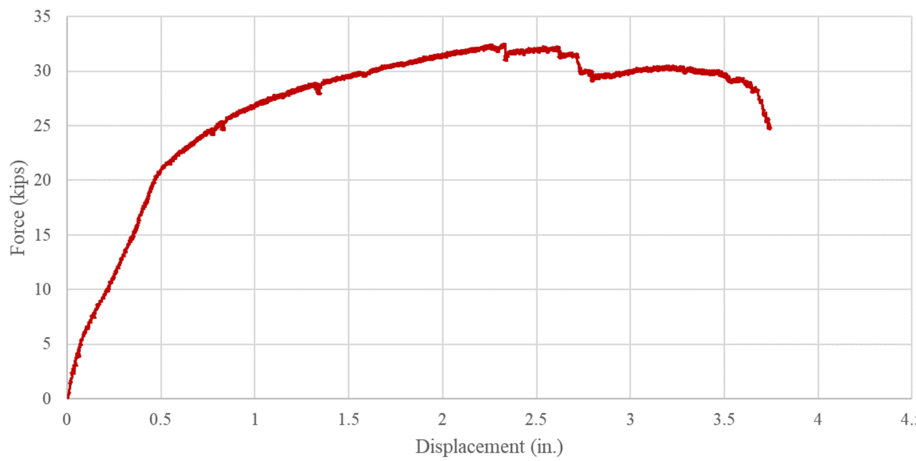


Figure 52: O-PR2 slab force-displacement graph.

a) Flexural cracks initially forming

b) Shear crack extending through the overlay

Figure 53: O-PR2 slab during testing.

6.4 O-PW (Overlay-Positive Weak)

The O-PW slab was similar to the previous two UHPC overlay slabs with the exception of the strength of the substrate concrete. In this specimen, the substrate concrete had a strength of 2.7 ksi. The force-displacement curve of this specimen is provided in Figure 54. Under loading, the first observed damage was the formation of flexural cracks at around 2.5 kips, again marked by the change in stiffness at around this load. It should be noted that this flexural crack formed at about half of what was observed for the previous two specimens, which is directly attributed to the significantly weaker substrate concrete. Again, this specimen continued to gain strength until the longitudinal reinforcement yielded, after which the stiffness of the specimen gradually decreased. Ultimately, a shear crack formed in the substrate concrete at a maximum load of 22.9 kips and a displacement of 2.6 inches. The shear crack propagated through the UHPC overlay on one side and along the interface on the other side (Figure 55); however, in this case, this propagation occurred almost instantaneously, and therefore there is a significant drop in capacity immediately after the formation of this initial shear crack.

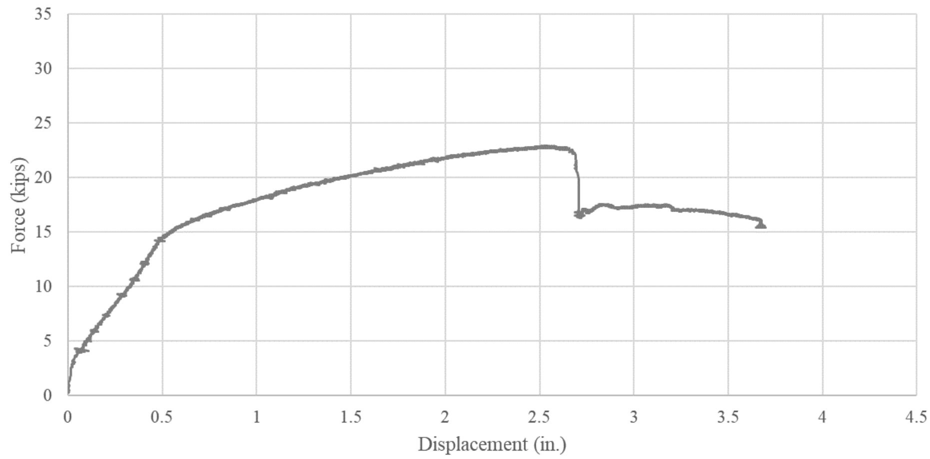


Figure 54: O-PW slab force-displacement graph.

a) Flexural cracks initially forming

b) Shear crack on one end of slab

c) Shear crack on other end of slab

Figure 55: O-PW slab during testing.

6.5 O-NR (Overlay-Negative Regular)

As discussed above, the O-NR slab was tested in a way to simulate a negative moment over the bridge girders. This specimen was tested using the same loading scheme used in the previous tests; however, the overlay was located on the bottom of the specimen, and the reinforcement was adjusted accordingly. The force-deflection curve for this test is provided in Figure 56, and Figure 57 shows the observed damage in the specimen. As expected, the behavior of this specimen was unique relative to the other specimens tested in this research. In this specimen, no flexural cracks formed immediately after loading due to the high tensile capacity of the UHPC overlay. Note the lack of a marked change in stiffness immediately after loading. That is, the initial stiffness is maintained until the UHPC begins cracking at around 20 kips, followed by a slight reduction in stiffness until reaching a peak load of 29.8 kips at a displacement of 0.55 inches. At which point, the UHPC cracks completely and there is a sharp loss of capacity, as observed in Figure 56. The capacity dropped to 18.0 kips, and then behaved very similar to the control specimen. This is expected since after the cracking of the UHPC the only thing carrying the load is the substrate concrete. The specimen ultimately failed at a deflection of 3.5 inches due to crushing of the concrete at the top of the specimen. It should be noted that this slab could fail with little to no warning since the post-cracked moment of this specimen is significantly less than its cracking moment. However, this situation may be acceptable if the strengthened capacity including the UHPC far exceeds the expected demand on the slab. Additionally, although ACI sets strict requirements for beam failures, the requirements are less stringent for slabs, as there is more inherent redundancy in the system. Design engineers should acknowledge this behavior when considering a UHPC bridge deck overlay.

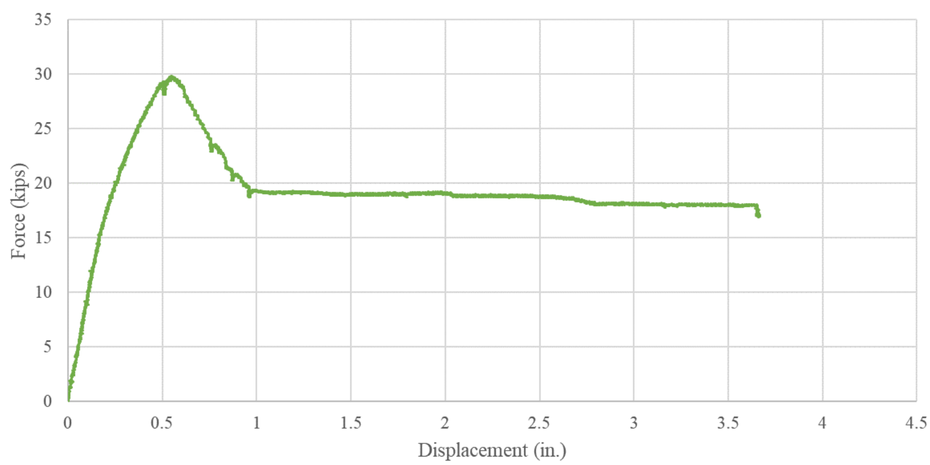
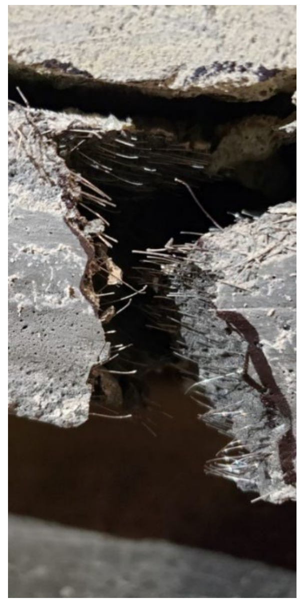


Figure 56: O-NR slab force-displacement graph.

a) Initial loading

b) Crack progressing through UHPC with steel fibers spanning



c) UHPC completely split

d) Midspan after failure, showing crushed concrete

Figure 57: O-NR slab during testing.

6.6 Discussion of Results

The following subsections discuss the key takeaways from this test series.

6.6.1 Overall Comparison and Observed Failure Mechanisms

All five force-displacement curves are shown in Figure 58, and Figure 59 is a plot of the resultant deflections along the length of all five specimens at a load of 12.5 kips. As can be observed in these plots, all specimens had similar initial stiffnesses prior to cracking, and all but the O-NR specimen had a marked reduction in stiffness below 5 kips due to flexural cracking. Note that the O-NR specimen did not crack completely until a load of nearly 30 kips due to the increased tensile capacity of the UHPC overlay, as will be discussed in greater detail below.

Regarding failure mechanisms, the Control slab failed due to concrete crushing in the compression zone at midspan, while the three positive moment slabs with overlays had initial failures due to shear in the substrate concrete. The fact that these specimens failed in shear rather than from concrete crushing is most likely due to the increased ultimate strain of UHPC and the subsequent delay in the onset of crushing. After the shear failures and subsequent drops in load, the three overlay specimens were able to partially recover, and hold load up to at least 3.5 inches of midspan deflection. It is important to note that although these specimens failed in shear, their overall behavior was still ductile due to the longitudinal steel yielding prior to failure. It should also be noted that although the overlay was observed to debond from the substrate concrete during shear failures, this apparent debonding is most likely due to dowel action on the longitudinal reinforcement, which is located at the interface between substrate concrete and UHPC overlay. More details will be discussed in a later section when the test results are compared to calculations.

As stated above, due to the much higher tensile capacity of the UHPC, the O-NR slab does not initially crack until a much higher load (~16 kips) compared to the other slabs (~4 kips). This result highlights the benefits that a UHPC overlay will have on both a large increase in stiffness and delaying the onset of initial surface cracking during negative bending. After the O-NR specimen reached a peak load of nearly 30 kips, there is a sudden drop in capacity when the tensile load is transferred solely to the longitudinal reinforcement. After this crack forms, the specimen behaves very similar to the control specimen, ultimately failing due to concrete crushing at the midspan.

The following subsections further discuss the effects that various parameters have on the overall performance of the slab specimens.

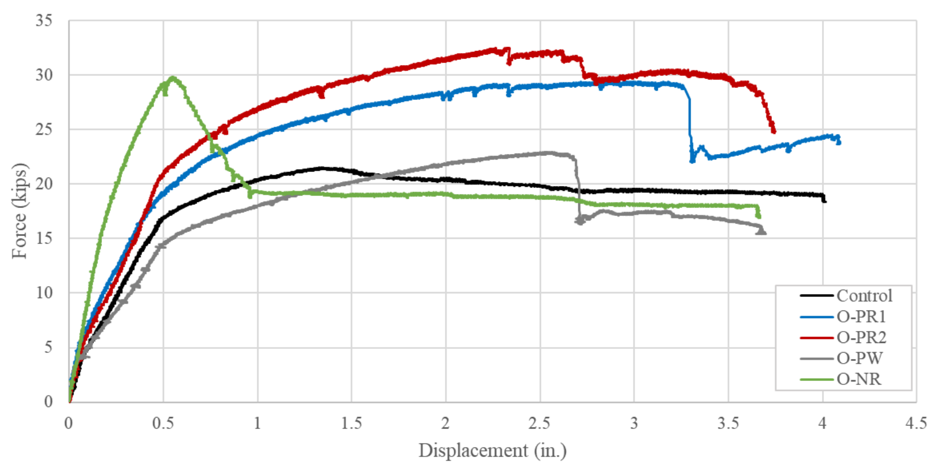


Figure 58: Comparison of all test slabs (force-displacement graphs).

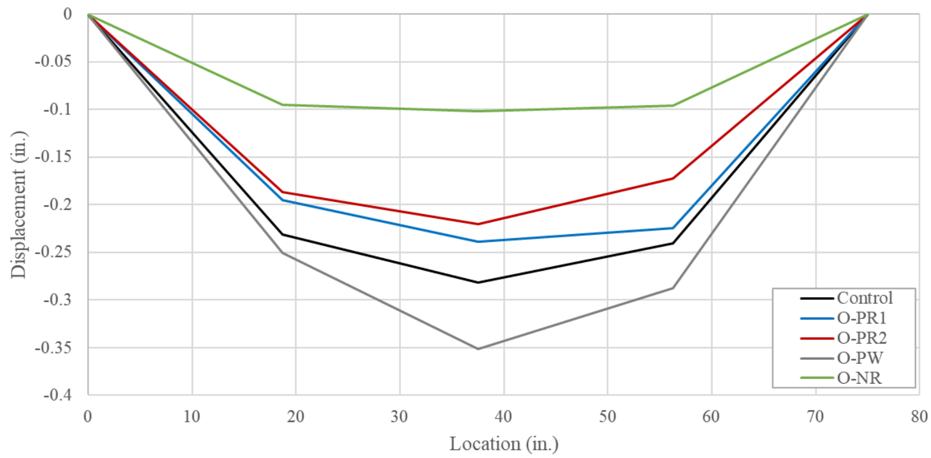


Figure 59: Deflections along the test span for all test slabs at 12.5 kips of load. Deflections were measured at 1/4 lengths (1/4, 1/2, and 3/4), or 18.75”, 37.5”, and 56.25” along the 75” span length.

6.6.2 Effects of UHPC Overlay Presence and Thickness

Figure 60 directly compares the force-displacement plots of the Control, O-PR1, and O-PR2 slabs to demonstrate the effect of replacing the top surface of the substrate concrete with a UHPC overlay. Both overlay specimens were stiffer and had higher ultimate capacities than the control specimen. Relative to strength, the O-PR1 and O-PR2 specimens were observed to be 37% and 51% stronger than the control specimen. These differences in stiffness and strength are most likely due to the UHPC overlay specimens being slightly deeper than the control specimen (6.51 in. and 6.64 in. vs. 6.28 in. for the control), and the fact that the UHPC is inherently stiffer and stronger than the conventional concrete in the control specimen. It should be noted, however, that concrete strength does not typically have a significant effect on capacity. That being said, an increase in compressive strength would decrease the size of the compression block at the top of the specimen, thus increasing the moment arm between the compression force and the tensile force. Further, UHPC has a higher ultimate strain than conventional concrete, thus delaying the onset of crushing in the overlay. This delay in concrete crushing could also explain the differences in the observed failure mechanisms. That is, the control specimen failed due to concrete crushing, while the delayed onset of crushing in the overlay specimens forced the failure mechanism into the substrate concrete where it failed due to shear.

Another comparison made on Figure 60 is between O-PR1 and O-PR2, demonstrating the effects that the UHPC overlay thickness has on the slab performances. O-PR1 and O-PR2 slabs had total depths of 6.51 in. and 6.64 in., respectively, and overlay thicknesses of 1.55 in. and 1.83 in., respectively. Therefore, as expected, the deeper specimen had a slightly higher capacity (10.5% stronger).

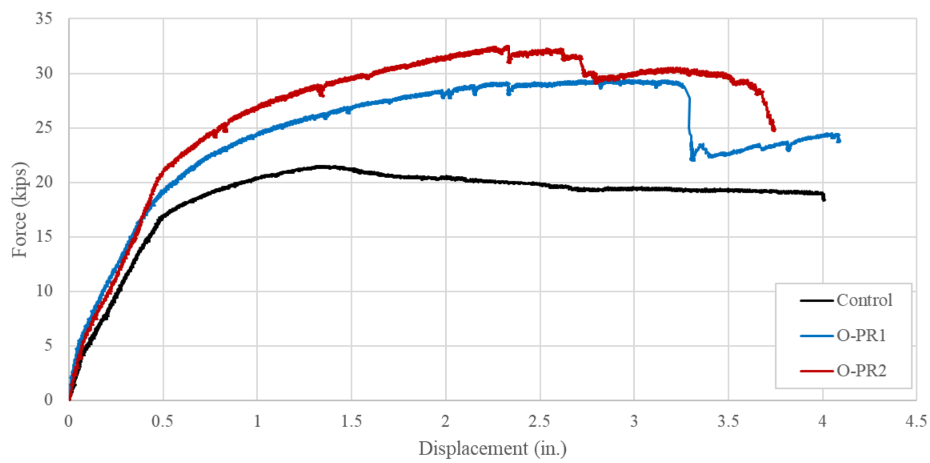


Figure 60: Control, O-PR1, and O-PR2 force- displacement graphs.

6.6.3 Effect of substrate concrete strength

Figure 61 directly compares the force-displacement plots of the Control, O-PR1, and O-PW specimens to demonstrate the effects that the substrate concrete compressive strength has on the flexural performance of the slabs. As expected, O-PR1 was stiffer and stronger than O-PW due to the increased stiffness and strength of the stronger concrete and due to the increased depth of this specimen. O-PR1 was 28.2% stronger than the O-PW specimen.

The Control and O-PW slabs performed similarly despite the large difference in substrate concrete strengths (4.1 ksi Control vs. 2.7 ksi O-PW) and total differences in overall depths (6.28 inches for Control vs. 5.73 inches for O-PW). Both specimens had similar stiffnesses and ultimate capacities were 21.5 kips and 22.9 kips, within 6.7%. This outcome demonstrates that a bridge deck made with weaker strength concrete can achieve a similar performance to a deck made with stronger conventional concrete when reinforced with a thin UHPC overlay. This finding holds significant potential for the rehabilitation of existing bridge decks made with weak concrete.

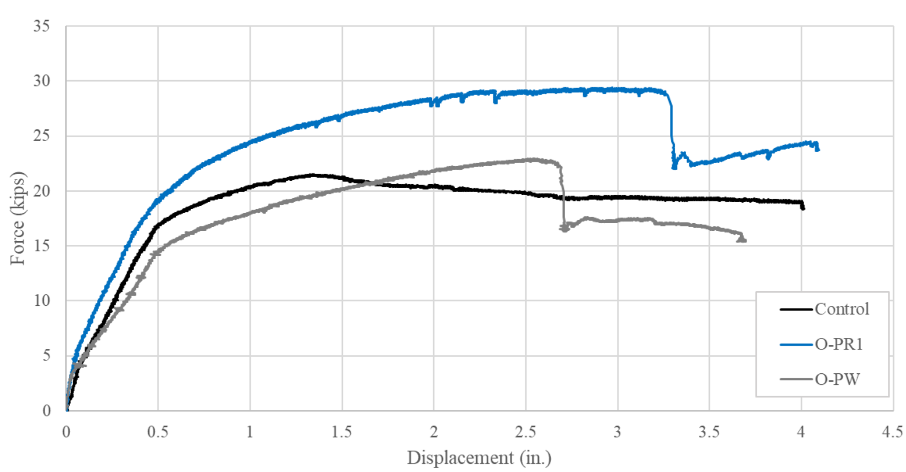


Figure 61: Control, O-PR1, and O-PW force-displacement graphs.

6.6.4 ACI/FHWA Predicted Capacities

The expected flexural and transverse shear capacities of the slab specimens were calculated according to methods prescribed ACI 318-19. Note that these calculations used the as-built dimensions and measured material properties on the day of testing, as presented in Section 4.6.

For flexural capacity, the Whitney stress block (with code-prescribed parameters) was used to model the concrete (both substrate and UHPC), and the steel was modeled as elastic-perfectly plastic. It should be noted that the mechanics in the negative moment specimen (O-NR) differ from those of the positive moment specimens, and these mechanics change throughout loading. That is, in this specimen and this loading configuration, the UHPC is at the bottom of the member in tension. The behavior of this specimen remained linear until the tensile capacity of the UHPC was reached, at which point the UHPC cracked and there was a sharp drop in capacity. After cracking, the UHPC has no effect on the behavior of the slab and the mechanics are then similar to that of the control specimen. For this specimen, the ultimate moment capacity is calculated according to the mechanics proposed by the FHWA [35], which accounts for the tensile capacity of the UHPC (Figure 62).

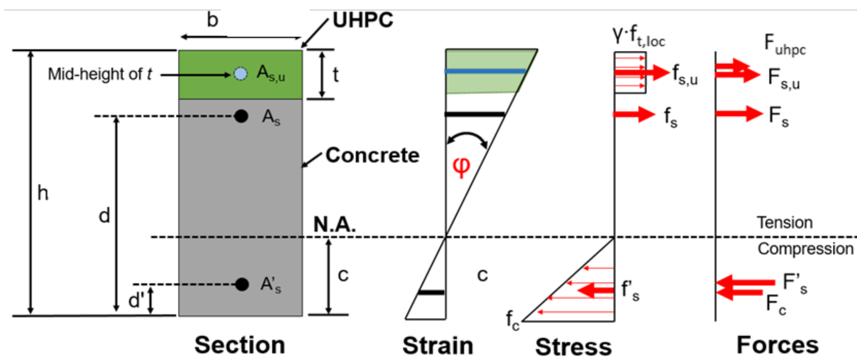


Figure 62: Assumed mechanics for negative moment capacity calculations, including UHPC tensile strength for UHPC bridge deck overlays [5].

The transverse shear capacity for each specimen was calculated as $V_c = 8\lambda_s \rho_w^{1/3} \sqrt{f_c'} b_w d$ (ACI 318-19), assuming the slab does not meet $A_{v,min}$, using the concrete compressive strength of the substrate concrete and ignoring the increased concrete strength of the UHPC. This assumption is conservative and is consistent with observed failures in which shear cracks initiated in the substrate concrete.

The predicted capacities using the methods discussed above are provided in Table 20, along with the measured capacities obtained from the tests. Included in this table are the ratios of measured to predicted capacities (Meas/Pred), and the observed failure mechanisms.

Table 20: Measured vs. Predicted Slab Capacities

Slab	Failure Mechanism	Measured Values			Moment Calculations		Shear Calculations	
		Measured P (kips)	Measured Moment (k-ft)	Measured Vc (kips)	Predicted Mn (k-ft)	Meas/Pred	Predicted Vc (kips)	Meas/Pred
Control	Concrete crushing	21.5	31.4	10.8	25.6	1.22	11.1	0.97
O-PR1	Shear in substrate	29.4	42.9	14.7	33.9	1.26	11.4	1.29
O-PR2	Shear in substrate	32.5	47.4	16.3	35.7	1.33	11.6	1.40
O-PW	Shear in substrate	22.9	33.4	11.5	28.4	1.18	10.2	1.12
O-NR	Cracking/ Concrete Crushing	29.8	43.5	14.9	45.4	0.96	10.9	1.37

For the positive moment specimens, the calculated flexural and shear capacities were close to the observed capacities and conservative. The ACI capacity calculations accurately predicted the expected failure responses in all positive specimens except in O-PW; however, the O-PW Meas/Pred ratios for moment and shear were very close.

As for the negative moment specimen, the FHWA predicted capacity (assuming a tensile stress of 3 ksi) was very close to the observed capacity, with a Meas/Pred ratio of 0.96. Note that for the negative moment specimen, these capacities are referring to the cracking capacity of the slab. As discussed previously, this specimen remained linear until the UHPC cracked in tension, leading to a shift in mechanics and ultimately a significant drop in capacity. This behavior (pre-crack capacity exceeding cracked capacity) could lead to a brittle failure with little to no warning. However, this situation may be acceptable if the cracking moment far exceeds the expected demand on the slab, and/or if the slab was designed to carry no more than the cracked capacity. It is also worth noting the calculated capacity of this specimen, ignoring the UHPC in tension, was determined to be 24.3 k-ft, which is close to the observed cracked capacity of 28 k-ft.

It should be noted that while some of the specimens showed signs of the UHPC overlay debonding, it was determined that this debonding was not associated with interface shear. As detailed in Appendix F, the calculated interface shear capacity was approximately 20 times higher than the interface shear demand on the specimens at the highest loads experienced by the slabs. The debonding was most likely attributed to dowel action on the longitudinal reinforcement initiated by the propagation of shear cracks.

6.7 Summary of Structural Testing

In this study, a series of tests were conducted on five reinforced concrete slab specimens to evaluate the performance of bridge decks with ultra-high-performance concrete (UHPC) overlays. The slab specimens were subjected to 3-Point flexural loading, simulating the loading conditions of bridge decks between longitudinal girders. The test series aimed to investigate the effects of various parameters, such as the presence of a UHPC overlay, overlay thickness, substrate concrete strength, and the difference between positive and negative moment behavior.

The test specimens included a control specimen without a UHPC overlay, two specimens with different UHPC overlay thicknesses (O-PR1 and O-PR2), a specimen with a lower substrate concrete strength (O-PW), and a specimen subjected to negative moment loading (O-NR). The slabs were instrumented to measure force and the subsequent deflections along their span. The results from the tests were analyzed in terms of stiffness, strength, failure mechanisms, and general behavior. Overall, the inclusion of a UHPC overlay significantly improved the stiffness and ultimate capacity of the positive moment slab specimens. The failure mechanisms of the positive moment slab specimens varied depending on the presence of a UHPC overlay, with the control slab failing due to concrete crushing in the compression zone at midspan and the three slabs with UHPC overlays failing due to shear in the substrate concrete. The specimen subjected to negative moment loading did not form significant flexural cracks until a much larger load compared to the other slabs. After this slab reached ultimate capacity, there was a significant drop in capacity, and the specimen behaved similarly to the control specimen and ultimately failed due to concrete crushing at the midspan. The control and weak substrate overlay slabs performed similarly despite the large difference in substrate concrete strengths; indicating that a bridge deck constructed with lower-strength concrete retrofitted with a thin UHPC overlay can exhibit comparable performance to a deck built with higher-strength conventional concrete.

The experimental results were also compared with the capacities predicted by the ACI 318-19 design code and the FHWA method for negative moment capacity calculations. Overall, regarding the efficacy of the capacity calculations, both ACI and FHWA predictions were in line with the test results and were mostly conservative.

7 Summary and Conclusions

The research project began with a literature review into applications of UHPC for bridge repair, primarily focusing on thin-bonded overlays and bridge member repair. Additional research was then conducted into completed UHPC bridge deck overlay projects and specifications from other states. Material level testing was then completed to investigate the workability, compressive and tensile strengths, and bond strengths with varying surface preparation techniques for three UHPC mixes. Structural testing was then completed on five slabs with UHPC overlays to quantify the effects that UHPC overlays have on the behavior and capacity of existing bridge decks. As stated previously, the structural testing portion of this research primarily focused on using a proprietary commercially available UHPC (Ductal) rather than the MT-UHPC. While MT-UHPC has been successfully used in several applications in the state, further research is needed prior to its use as a bridge deck overlay.

Specific conclusions from the material level evaluation (Chapter 3) and the structural testing (Chapters 5 and 6) are presented in the following sections.

7.1 Conclusions from the Material Level Evaluation

- The three UHPC mixes tested in this research had adequate compressive and tensile strengths, in line with previous research on UHPC. The MT-UHPC conventional mix and the Ductal-T mix had 28-day compressive and tensile strengths of around 17 ksi and 3.4 ksi, respectively. The thixotropic MT-UHPC had slightly less strength at 28 days, with compressive and tensile strengths of around 15 ksi and 2.8 ksi, respectively. While these strengths were slightly less, it is important to note that this was the first large-scale batch of this material, and higher strengths may be acquired if this mix is refined.
- The two thixotropic mixes investigated in this research (MT-UHPC-T and Ductal-T) had appropriate flows for the desired overlay application, where a stiffer mix is required for placement on graded/crowned bridges. The MT-UHPC-T had static and dynamic flows of 4" and 5.5", while the Ductal-T mix had static and dynamic flows of 4" and 6.5". The dynamic flow of the MT-UHPC-T mix is slightly low, but again this is the first large-scale batch of this material, and better flows may be acquired with some refinement.
- The direct-tension bond tests for all three concretes and nearly all surface preparation methods reached the minimum strength specified by ACI for concrete repairs. The only specimens that did not meet this minimum were the Ductal-T specimens in which the surface of the substrate concrete was not wetted prior to placement of the UHPC overlay, highlighting the importance of this step.
- The minimum bond strengths obtained from the slant-shear tests for all concretes met the ACI specified shear bond for concrete repairs. This, despite a conservative surface preparation method with grooves parallel to the loading direction. Also, it is important to point out that all but one specimen failed due to concrete crushing in the substrate concrete, and therefore the actual bond stresses at failure were not obtained and the recorded values can be interpreted as minimum values.
- For many of the direct-tension tests and nearly all of the slant-shear tests the specimens failed in the substrate concrete prior to the bond failure, and therefore the recorded bond strengths can be interpreted as minimum values. Future research could modify these tests to ensure failure in the

bond. For example, the substrate concrete in the slant shear tests could be wrapped with FRP prior to testing to ensure that this concrete does not fail prematurely in compression.

7.2 Conclusions from the Structural Testing

- The inclusion of a UHPC overlay significantly improved the stiffness and ultimate capacity of the positive moment slab specimens. The O-PR1 and O-PR2 specimens exhibited 37% and 51% higher strengths, respectively, compared to the control specimen. Further, it was observed that an increase in UHPC overlay thickness resulted in a 10.5% increase in strength for the O-PR2 specimen compared to the O-PR1 specimen.
- The observed failure mechanisms of the positive moment slab specimens varied depending on the presence of a UHPC overlay and other influencing factors. The Control slab, without a UHPC overlay, failed due to concrete crushing in the compression zone at midspan. In contrast, the three slabs with UHPC overlays (O-PR1, O-PR2, and O-PW) ultimately failed due to shear in the substrate concrete. This difference in failure mechanisms can be attributed to the increased ultimate strain of UHPC, which delayed the onset of concrete crushing in the overlay specimens. This delay in crushing forced the failure mechanism into the substrate concrete, where it failed due to shear, well after the longitudinal steel had yielded and there were significant deflections.
- The O-NR specimen, subjected to negative moment loading, did not form significant flexural cracks until a load of nearly 30 kips, owing to the high tensile capacity of the UHPC overlay. After this crack formed, there was a significant drop in capacity, and the specimen behaved similarly to the control specimen and ultimately failed due to concrete crushing at the midspan. It should be noted that this behavior (pre-crack capacity exceeding cracked capacity) could lead to a brittle failure with little to no warning. However, this situation may be acceptable if the cracking moment far exceeds the expected demand on the slab, and/or if the slab was designed to carry no more than the cracked capacity. There is inherent redundancy in a slab system, but design engineers should acknowledge this potential brittle behavior when considering a UHPC bridge deck overlay.
- The Control and O-PW slabs performed similarly despite the large difference in substrate concrete strengths (4.1 ksi Control vs. 2.7 ksi O-PW) and differences in overall depths; indicating that a bridge deck constructed with lower-strength concrete retrofitted with a thin UHPC overlay can exhibit comparable performance to a deck built with higher-strength conventional concrete.
- Regarding the efficacy of capacity calculations, the ACI calculations were in line with the test results for the positive moment specimens and were conservative, with Meas/Pred ratios ranging between 1.12-1.40. For the negative moment specimen, the FHWA predicted capacity closely matched the observed capacity, with a Meas/Pred ratio of 0.96, when accounting for the tensile capacity of the UHPC.

7.3 Concluding Remarks

Overall, UHPC overlays were shown to be a potential option for retrofitting existing bridge decks, as they significantly improved the performance of the concrete slabs, allowing for increased strength and durability. However, the specific benefits should be evaluated on a case-by-case basis, taking into account factors such as cost, construction constraints, and the specific demands of the project. To further capitalize on the findings of this research, an implementation project employing a UHPC bridge-deck overlay on a bridge in Montana could be pursued in a future project. Additionally, to make possible the use of MT-UHPC as a bridge deck overlay material, further research could be conducted to refine/enhance its thixotropic properties and to increase batch sizes.

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Appendix A: MasterMatrix UW 450 Spec Sheet

3 | 03 30 00 Cast-in-Place Concrete
03 37 26 Underwater Placed Concrete

MASTER® BUILDERS SOLUTIONS

MasterMatrix® UW 450

Anti-Washout Admixture

Description

MasterMatrix UW 450 anti-washout admixture is a patented, ready-to-use, liquid cellulose-based admixture that is specially developed for underwater concrete applications. Concrete containing MasterMatrix UW 450 admixture exhibits superior resistance to washout of cement and fines, while impeding the blending of external water into the plastic concrete.

MasterMatrix UW 450 admixture meets the requirements of the U.S. Army Corps of Engineers CRD-C661-06,Specification for Anti-Washout Admixtures for Concrete.

Applications

Recommended for use in:

- All types of underwater concreting where conventional concrete or placing techniques would result in a high percentage of material loss due to washout
- Mortar and grouting applications where mixtures are typically more fluid and have a higher potential for washout

Features

- Reduction in washout of cement and fines
- Reduction in segregation, even with highly fluid, high water-to-cementitious materials ratio concrete mixtures
- Thixotropic action that provides concrete stiffening after placement
- Reduction or elimination of concrete bleeding

Benefits

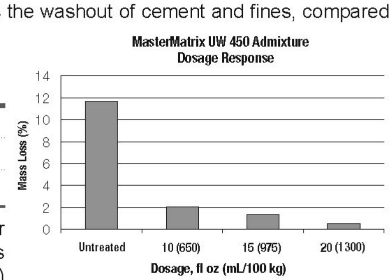
- Superior and predictable in-place concrete properties
- Dewatering costs reduced/eliminated
- Environmental impact of cement washout in water minimized
- Flexibility in batching procedures

Performance Characteristics

Washout Resistance: Washout is determined by Army Corps of Engineers CRD-C 61, “Test Method for Determining the Resistance of Freshly Mixed Concrete to Washing Out in Water”. Test results show that the addition of MasterMatrix UW 450 anti-washout admixture to concrete significantly reduces the washout of cement and fines, compared to untreated concrete.

Concrete Mixture Data (Non-Air-Entrained Concrete)

Cement Content	650 lb/yd^3 (386 kg/m^3)
Water-Cement Ratio	0.49
Slump	4 ± 0.5 in. (100 ± 10mm)



Slump: Concrete that is designed for underwater placement applications is typically batched at an 8-10 in. (200-250 mm) slump. After MasterMatrix UW 450 admixture is added, a decrease in slump will be noted. It may be necessary to add additional high-range water-reducing admixture to achieve the slump required for placement. Slump evaluations for a 60-minute period show that MasterMatrix UW 450 admixture does not adversely affect concrete slump retention.

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MasterMatrix UW 450 Technical Data Sheet

Air Content: A slightly higher dosage of air-entraining admixture may be required to achieve the desired air content when using MasterMatrix UW 450 admixture.

Setting Time: MasterMatrix UW 450 admixture has little to no effect on concrete setting time at commonly used dosages of 4-12 fl oz/cwt (260-780 mL/100 kg). Slight retardation of setting time may be experienced at dosages over 12 fl oz/cwt (780 mL/100 kg).

Compressive Strength: Using test specimens that are cast in air, concrete containing MasterMatrix UW 450 admixture may obtain slightly lower compressive strength when compared to untreated concrete. However, when strength is evaluated using test specimens that are cast underwater, concrete containing MasterMatrix UW 450 admixture achieves higher strength because washout is minimized. In addition, most underwater concrete mixtures that are proportioned in accordance with ACI 304R, "Guide for Measuring, Mixing, Transporting, and Placing Concrete", exceed compressive strengths that are required for underwater applications. If necessary, a lower water-to-cementitious materials ratio may be used to achieve the desired results.

Guidelines for Use

Dosage: MasterMatrix UW 450 admixture is recommended for use at a dosage range of 4-20 fl oz/cwt (260-1300 mL/100 kg) of cementitious materials for most concrete mixtures. Because of variations in concrete materials, jobsite conditions and/or applications, dosages outside of the recommended range may be required.

Mixing: For underwater concrete placements, ACI 304R, Chapter 8, "Concrete Placed Underwater" provides certain basic mixture proportions such as:

- A minimum total cementitious material content of 600 lb/yd^3 (356 kg/m^3)
- Use of pozzolans approximately 15% by mass of cementitious materials
- A maximum water-to-cementitious materials ratio of 0.45
- Fine aggregate contents of 45-55% by volume of total aggregate
- Air contents of up to 5% are listed as desirable
- A slump of 6-9 in. (150-230 mm) is generally necessary and occasionally a slightly higher slump range is needed

MasterMatrix UW 450 admixture should be added with a water-reducing admixture, such as Master Builders Solutions MasterPolyheed® or MasterSet® admixture lines. For achieving high slump concrete, use MasterMatrix UW 450 admixture in conjunction with a MasterGlenium® high-range water-reducing admixture. This combination will produce a high-performance, flowing concrete that exhibits superior resistance to washout of cement and fines. MasterMatrix UW 450 admixture should be added after all other concreting ingredients have been batched and thoroughly mixed, either at the batch plant or at the jobsite.

Concrete Placement: Concrete containing MasterMatrix UW 450 admixture is easily pumped throughout the typical slump ranges that are used for underwater concreting. It is recommended that concrete containing MasterMatrix UW 450 admixture is placed by pump or tremie. Concrete placement should be continuous and without interruption. Keep the discharge point of the placement device immersed in the fresh concrete during placement.

It is not recommended that concrete containing MasterMatrix UW 450 admixture be allowed to free-fall through water during placement.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: MasterMatrix UW 450 admixture will neither initiate nor promote corrosion of reinforcing and prestressing steel embedded in concrete, or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of this admixture.

Compatibility: Do not use MasterMatrix UW 450 admixture with naphthalene-based high-range water-reducing admixtures. Erratic behaviors in slump, pumpability and washout may be experienced.

Storage and Handling

Storage Temperature: MasterMatrix UW 450 admixture must be stored at temperatures above 44 °F (7 °C) to avoid dispensing difficulties due to thickening. Do not allow MasterMatrix UW 450 admixture to freeze since it cannot be reconstituted after thawing.

Shelf Life: MasterMatrix UW 450 admixture has a minimum shelf life of 12 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of MasterMatrix UW 450 admixture has been exceeded.

Handling: Contact with water in hoses, pumps, tanks or receiving vessels must be avoided to prevent gelling when transferring MasterMatrix UW 450 admixture to other containers.

Dispensing: Consult your local sales representative for the proper dispensing equipment for MasterMatrix UW 450 admixture. If dispensing directly from the 55 gal (208 L) drum, it is recommended that the larger 2 in. (50 mm) opening be used.

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MasterMatrix UW 450 Technical Data Sheet

Packaging

MasterMatrix UW 450 admixture is supplied in 53 gal (201 L) drums and 264 gal (999 L) totes.

Related Documents

Safety Data Sheets: MasterMatrix UW 450 admixture

Additional Information

For additional information on MasterMatrix UW 450 admixture or its use in developing concrete mixtures with special performance characteristics, contact your local sales representative.

Master Builders Solutions, a brand of MBCC Group, is a global leader of innovative chemistry systems and formulations for construction, maintenance, repair and restoration of structures. The Admixture Systems business provides advanced products, solutions and expertise that improve durability, water resistance, energy efficiency, safety, sustainability and aesthetics of concrete structures, above and below ground, helping customers to achieve reduced operating costs, improved efficiency and enhanced finished products.

Utilizing worldwide resources, the Master Builders Solutions community of experts are passionate about providing solutions to challenges within all stages of construction, as well as the life cycle of a structure. At Master Builders Solutions we create sustainable solutions for construction around the globe.

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Appendix B: FHWA Recommendations

The following bullets summarize the specified construction considerations from FHWA [34]:

General UHPC Recommendations

- **Development length**
  - For deformed steel reinforcement No. 8 and smaller, the embedment length should be equal to or greater than the development length, l_d
    - When cover $\geq 3d_b$:
      - $l_d \geq 8d_b$ for reinforcing bars with yield strength $f_y \leq 75$ ksi
      - $l_d \geq 10d_b$ for reinforcing bars with yield strength 75 ksi $\leq f_y \leq 100$ ksi
    - When $2d_b \leq$ cover $\leq 3d_b$
      - Increase minimum l_d by $2d_b$
  - For concrete bridge deck applications, the embedment length of No. 5 deformed bars can be taken as:
    - When cover ≥ 1.25 inches:
      - $l_d \geq 8d_b$ for reinforcing bars with yield strength $f_y \leq 75$ ksi
    - When 1.0 inch $\leq$ cover < 1.25 inches:
      - $l_d \geq 10d_b$ for reinforcing bars with yield strength $f_y \leq 75$ ksi
- **Lap splices**. The lap splice length, l_s
  - $l_s \geq 0.75\ l_d$
  - Clear spacing to nearest spliced bar $\leq l_s$
- **Minimum cover and spacing** of reinforcing bars should not be less than the greater of
  - 1.5 times the longest fiber length included in the UHPC
  - 0.75 inch

  (Unless adequate fiber distribution is otherwise demonstrated for a specific application.)
- **Formwork and traffic vibration mitigation**
  - Watertight
  - Able to withstand hydrostatic pressures from UHPC and buoyancy forces on any top forms
  - Surfaces should be nonabsorbent (oiled, resin coated, plastic wrapped plywood, steel, etc.) to avoid pulling moisture from the UHPC
  - External vibration from traffic and removal of formwork should be avoided until 14 ksi strength is achieved

- **Mixing**
  - Batch and mix according to developer or manufacturer recommendations
  - Store materials and mix water at reduced temperatures
  - Target higher end of flow range for hot weather, and lower end of flow range for cold weather
  - Mixer should be capable of dispersing liquids and fibers uniformly. Typically, most concrete or grout mixers can be used for mixing, but at one-third to two-thirds the volume.
  - Temperature of UHPC at end of mix should be kept between 40 and 80 °F. Ice can be used to replace some or all mix water.
- **Placement and consolidation**
  - Fresh UHPC should be transported, placed, and covered as soon as possible.
  - Thixotropic UHPC should be vibrated as necessary for good consolidation though constituent segregation should be avoided.
- **Curing and strength gain**
  - Should be protected from freezing until minimum 14 ksi compressive strengths
  - Exposure to the external environment should be avoided until specified minimum strength is achieved. This can be achieved through a combination of conventional concrete curing compound and plastic sheeting.

UHPC Overlay Recommendations

Most points discussed above for general UHPC recommendations apply to UHPC bridge deck overlays and additional points specific to overlays are listed here:

- **Material consistency**
  - Thixotropic such that it can be placed without top forming
- **Fiber content**
  - Should be based on mechanical properties for strength and serviceability objectives; however, most overlays to date used 3.25% by volume
- **Thickness, clear spacing, and cover**
  - Minimum finished overlay thickness should be the greater of
    - 1.0 inch
    - Or 1.5 times the max fiber length
  - Minimum nominal clear cover after finishing and profiling over reinforcing bars should be 0.625 inch

  - Minimum clear distance between reinforcing bars and existing concrete deck substrate should be greater of 0.5 inch or the maximum fiber length

- **Existing deck concrete substrate preparation**
  - Concrete substrate should be roughened to a minimum profile of 0.125 inch, measured as the average value between peaks and valleys or equivalent to a roughness average (Ra) of 0.0625 as defined by ASME B46.1.
  - Substrate surfaces should be roughened with both macro- and micro-texture to enhance the bond strength. Micro-texture is more important for tensile bond strength and is best achieved by removing the cement paste. Macrotexture is more important for shear strength.
  - UHPC to UHPC can be bonded with set retarders to expose the fibers.
  - Prewetting concrete to SSD is very important and typically requires a minimum of 6 hours or more of continuous wetting.
  - Bonding agents can be considered, although no long-term data on performance has been gathered.
- **Skid Resistance**
  - Completed surface must provide adequate skid resistance. The desired surface is typically achieved through grinding the entire surface. Texturing the surface is also allowed; however, skid resistance must be validated, such as with ASTM E303.
- **Phased construction joints**
  - Construction joints should be detailed to maximize bond, minimize water, and provide mechanical continuity. Joints should be reinforced if placed in negative bending region. May not be necessary if existing deck reinforcement is fully encapsulated in UHPC. UHPC fibers should be exposed.
- **Existing deck surface preparation**
  - For deep removals or areas that are heavily patched, scarifying should be performed first for a uniform removal, then followed by Hydrodemolition or sand blasting. Hand chipping should be avoided (to avoid microcracks), but when necessary limited to a hammer size of 35 pounds maximum.
- **Placing and finishing equipment and methods**
  - Conventional concrete deck screeds can be used to spread, consolidate, and finish overlays less than 2 inches; however, if a thixotropic mix is used, the placement still requires significant assistance in distributing the material evenly before a pass is made with the screed.
  - Automated bridge deck finishing machines designed specifically for UHPC have been used on numerous overlay installations in the United States and should be considered for

thixotropic mixes. This is because conventional concrete bridge deck finishing machines have issues with the UHPC sticking to the augers and rollers and UHPC surface tearing.

- **Postconstruction concerns**
    - Exposed fibers should not be a concern. Fibers have not shown to cause any damage to vehicles, pedestrians, or animals. Over time the fibers will rust. Eventually the fibers and rust will disappear.

Appendix C: Placement Plan

The following text is directly from the Iowa Special Provision for UHCP Overlays [36], detailing the placement plan.

1. Submit a Placement Plan with a detailed construction work schedule to the Engineer for review and approval at least 30 days prior to the scheduled UHPC placement pour. The following list is intended as a guide and may not address all of the means and methods the contractor may elect to use. The Contractor is expected to assemble a comprehensive list of all necessary items for executing the placement of UHPC.
    - Responsible personnel and hierarchy.
    - Equipment – including but not limited to mixers, holding tanks, generators, wheelbarrows, scales, meters, thermometers, floats, screeds, burlap, plastic, heaters, blankets, etc.
    - Quality Control of batch proportions - including dry ingredients, steel fibers, water and admixtures.
    - Quality Control of mixing time and batch times.
    - Batch procedure sequence.
    - Form work – including materials and removal.
    - Placement procedure – including but not limited to surface preparation of existing concrete surfaces and pre-wetting of the existing concrete interface to a saturated surface-dry (SSD) condition before the placement of UHPC), spreading, finishing, and curing protection. Include provisions for acceptable ambient conditions and batch temperatures and corrective measures as appropriate.
    - Threshold limits for ambient temperature, ambient relative humidity, batch consistency, batch temperature, batch times and related corrective actions.
2. A preconstruction meeting will be held between the UHPC manufacturer's representative, the Contractor's staff, and representatives from Iowa DOT District Office, Office of Bridges and Structures, and Office of Construction and Materials to review the Contractor's Placement Plan prior to placement of UHPC materials. No UHPC pour will be permitted until the aforementioned Placement Plan has been submitted by the contractor and approved by the Engineer.
3. Pumping of UHPC is not allowed.
4. Construction loads applied to the bridge during UHPC placement and curing are the responsibility of the contractor. Submit the weight and placement of concrete buggies, grinding equipment or other significant construction loads for review as part of the proposed Placement Plan.

Appendix D: MasterFinish XR Spec Sheet

3 | 03 30 00 Cast-in-Place Concrete
03 40 00 Precast Concrete

MASTER® BUILDERS SOLUTIONS

MasterFinish® XR

Water-based retarder for exposed-aggregate concrete

Formerly MBT X-Poz-R*

Description

MasterFinish XR is a water-based top surface and/or form retarder used to produce uniform exposed-aggregate finishes on architectural or ornamental concrete.

Applications

Recommended for use in:

- Architectural concrete
- Ornamental concrete
- Face-up exposed-aggregate applications
- Steps
- Construction joints

Location

- Horizontal

Substrate

- Steel forms
- Fiberglass forms
- Concrete forms
- Wood forms
- Styrofoam forms
- Urethane form liners
- Fresh concrete

Features

- Versatile
- Excellent for use on Styrofoam or urethane form liners
- Ease of application
- Economical
- Wide range of etches available
- No need to wash off top surface on the same day
- Water-based

Benefits

- Can be used on a wide variety of form materials on small applications
- Extends the life of the forms
- Excellent top surface results
- No mold release needed
- Variety of exposed aggregate appearances
- Easy for scheduling labor
- No solvent, no odor; no fume or flammability problems

Color

The material in every container of MasterFinish XR retarder is color-coded to show specific degrees of penetration. Each container bears the name of the color on the label. The lid is also colored to match the material inside.

Color Coding Information

Color (Code)	Depth of Etch	Aggregate Size* in. (mm)
Lilac (38)	Light	1/4 – 3/8 (6 – 10)
Pink (48)	Medium Light	3/8 – 1/2 (10 – 13)

*Recommended aggregate size

A brand of MBCC GROUP

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Guidelines for Use

Yield: 150 - 200 ft^2/gal (3.7 - 4.9 m^2/L).

Surface Preparation: Forms must be clean, dry and non-porous. Seal porous forms of wood or concrete with MasterFinish PT 100 protective coating, specially formulated to use under a retarder. Once MasterFinish XR retarder has been applied to the form or formliner, it must be protected against rain, condensation, accidental contact with water, etc.

Mixing: Mix thoroughly before use with a mechanical mixer. The retarder can also be mixed by pouring back and forth between buckets.

Application:

1. Apply MasterFinish XR retarder in a thin uniform coat by spray, brush or roller at a rate of 150 - 200 ft^2/gal (3.7 - 4.9 m^2/L) (8 mils (0.2 mm) on wet film thickness gauge). This coverage should be achieved by applying two light coats with a short (3/8 in. (10 mm)) nap roller.
2. Allow the first coat to dry for 30-60 minutes or until dry to touch before applying second coat.
3. The second coat should be applied perpendicular to the first coat with light roller pressure (too thick a coating is undesirable).
4. After the second coat is dry (approximately 30-60 minutes), the concrete or face mix may be placed. If the placing of concrete is delayed any unusual length of time, protect the retarder surface from rain or water.
5. When the element is lifted or the forms are removed, the retarded surface concrete is easily removed by sandblasting, high pressure water-washing or dry-brushing.

Top Surface Application:

1. Paint sprayer must be used. MasterFinish XR retarder may be thinned with water up to 20% to accommodate spraying, but must be mixed thoroughly.
2. Place the concrete and screed, float or trowel level. Do not tamp or overwork the top surface as this drives the aggregate down away from the top surface and thereby diminishes the depth and uniformity of exposure.
3. Use a wood float or bull float to work the top surface. Do not overwork the surface with a steel trowel as this tends to seal the top surface thereby restricting the penetration of the chemical retarder, thus restricting the penetration of the MasterFinish XR retarder.
4. As soon as surface water has disappeared (normally no more than 30 minutes), spray until the color of the retarder just "hides" the area to be exposed. Protect areas adjacent to the job from overspray which may stain these areas. Apply at the rate of 150- 200 ft^2/gal (3.7 – 4.9 m^2/L) (8 mils (0.2 mm) on wet film thickness gauge.)

To Expose Aggregate:

1. Under normal conditions, after the concrete has set overnight, direct a jet of water from a garden hose over the surface of the slab while scrubbing with a coarse floor brush to remove the retarded concrete paste.
2. The concrete not in contact with MasterFinish XR retarder will set firmly to resist erosion; the embedded exposed aggregates will hold securely when the concrete has fully cured.
3. Since retarded concrete will eventually harden, the surface should be checked periodically to determine degree of etch by washing a small area. When the temperature is high, or high early cement is used, check the surface earlier.
4. If the depth is excessive, wait a couple of hours and check the etch again.

Clean Up: Clean all tools and spray equipment with soap and water immediately after use.

MasterFinish XR Technical Data Sheet

For Best Performance

- Not suitable for large panels (larger than 4 ft by 4 ft (1.2 m by 1.2 m)).
- MasterFinish XR retarder will freeze at low temperatures. Allow to thaw and mechanically mix to bring back to original consistency.
- In conditions of high ambient temperatures or wind, concrete should be covered with black plastic sheeting.
- Recommended maximum concrete temperature is 110 °F (43 °C).
- Do not store in metal spray equipment more than 2-3 hours or in direct sunlight.
- Make certain the most current versions of product data sheet and safety data sheet are being used; call your local sales representative to verify the most current versions.

Mock Up

- Skill and practice are necessary to produce any high quality architectural finish. Samples and mock-ups duplicating actual production conditions are essential to obtain a representative finish for approval prior to commencing production.
- Some important variables that should be controlled as close to actual cast conditions include: retarder coverage rate and method of application, mixture proportions and slump, admixtures, temperature of plastic and cured concrete, vibration, thickness of the element, length of time in form and method of cleaning. This is especially important with light etches which are particularly affected by changing conditions.
- Changes in mixture proportions (cement, sand, aggregate, water), admixture content, temperature and any other factor influencing compressive strength development should be kept to a minimum. If white cement (Type III) is used in the face mix, consult your local sales representative.

Storage and Handling

Storage Temperature: Store in unopened container in a cool, clean, dry area between 40 and 110 °F (4 and 43 °C). Keep containers tightly sealed after opening to maintain shelf life freshness. **Protect from freezing.**

Shelf Life: 2 years when properly stored.

In case of spillage, clean and dispose of in accordance with local, state and federal applicable regulations.

Packaging

5 gal (18.9 L) pails

230 gal (870 L) totes

VOC Content

10 g/L or 0.08 lb/gal, less water and exempt solvents.

Related Documents

Safety Data Sheets: MasterFinish XR retarder

Additional Information

For suggested specification information or for additional product data on MasterFinish XR retarder, contact your local sales representative.

Master Builders Solutions, a brand of MBCC Group, is a global leader of innovative chemistry systems and formulations for construction, maintenance, repair and restoration of structures. The Admixture Systems business provides advanced products, solutions and expertise that improve durability, water resistance, energy efficiency, safety, sustainability and aesthetics of concrete structures, above and below ground, helping customers to achieve reduced operating costs, improved efficiency and enhanced finished products.

Utilizing worldwide resources, the Master Builders Solutions community of experts are passionate about providing solutions to challenges within all stages of construction, as well as the life cycle of a structure. At Master Builders Solutions we create sustainable solutions for construction around the globe.

www.master-builders-solutions.com/en-us page 3 of 4

MasterFinish XR

Technical Data Sheet

Limited Warranty Notice

Master Builders Solutions Admixtures US, LLC ("Master Builders Solutions") warrants this product to be free from manufacturing defects and to meet the technical properties on the current Technical Data Guide, if used as directed within shelf life. Satisfactory results depend not only on quality products but also upon many factors beyond our control. MASTER BUILDERS SOLUTIONS MAKES NO OTHER WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE WITH RESPECT TO ITS PRODUCTS. The sole and exclusive remedy of Purchaser for any claim concerning this product, including but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is shipment to purchaser of product equal to the amount of product that fails to meet this warranty or refund of the original purchase price of product that fails to meet this warranty, at the sole option of Master Builders Solutions. Any claims concerning this product must be received in writing within one (1) year from the date of shipment and any claims not presented within that period are waived by Purchaser. MASTER BUILDERS SOLUTIONS WILL NOT BE RESPONSIBLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDING LOST PROFITS) OR PUNITIVE DAMAGES OF ANY KIND.

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Appendix E: WR Meadows 1600 White Product Data

PRODUCT DATA

W. R. MEADOWS.
SEALTIGHT.

MasterFormat: 03 39 23

NO. 373

AUGUST 2016
(Supersedes August 2008)

1600-WHITE

Wax-Based, Water Emulsion Concrete Curing Compound

DESCRIPTION

The 1600-WHITE series of water-based, white-pigmented concrete curing compounds are wax-based dispersions with selected white pigments. When properly applied, 1600-WHITE forms a premium-grade membrane, which optimizes water retention. The white pigment reflects the sun's rays to help keep the concrete surface cooler and prevent excessive heat buildup.

USES

1600-WHITE is ideal for application on exterior, horizontal surfaces, such as highways, airports, and street and curb paving. 1600-WHITE is not recommended for residential applications. The product provides optimum curing when protection from the sun's heat is desired.

FEATURES/BENEFITS

- When properly applied, provides a premium-grade film, which optimizes water retention.
- Protects by reflecting the sun's rays to keep the concrete surface cooler and prevent excessive heat buildup, which can cause thermal cracking.
- Furnished as a ready-to-use, true water-based compound.
- Produces hard, dense concrete ... minimizes hair checking, thermal cracking, dusting, and other defects.
- Offers a compressive strength significantly greater than improperly or uncured concrete.
- Increases tensile strength for greater resistance to cracking and surface crazing.
- Improves resistance to the abrasion and corrosive actions of salts and chemicals ... minimizes shrinkage.
- Applies quickly and easily with conventional commercial spray equipment.
- VOC compliant.

PACKAGING

5 Gallon (18.93 L) Pails
55 Gallon (208.20 L) Drums

COVERAGE

Approximately 200 ft.^{2}/gal. (4.91 m^2/L). Coverage rates may have to be adjusted in windy conditions.

SHELF LIFE

When stored indoors in original, unopened containers at temperatures between 40° - 90° F (4° - 32° C), optimum performance and best use is obtained within one year of date of manufacture.

TECHNICAL DATA

VOC Content: 59 g/L

SPECIFICATIONS

- ASTM C 309, Type 2, Class A
- AASHTO M 148, Type 2, Class A
- FAA Spec. Item P-610-2.11 (e)
- Complies with all current federal, state, and local maximum allowable VOC requirements, including U.S. EPA, LADCO, SCAQMD, and OTC.

APPLICATION

Surface Preparation … Application equipment must be clean and free of any previously used materials.

Mixing … Any settling or separation in the container must be re-dispersed with gentle agitation prior to use. CAUTION: TO AVOID FOAMING, DO NOT MIX EXCESSIVELY. DO NOT THIN.

Application Method … Spray on in one even coat with a hand or power sprayer, such as a Chapin 1949, as soon as the surface water disappears from concrete surface. Use a spray nozzle that produces a flow of 0.5 GPM (1.89 LPM) under 40 psi (.276 MPa) of pressure.

CONTINUED ON REVERSE SIDE…

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Drying Time … Typically dries in two hours, depending on jobsite conditions (temperature, wind, etc.). Restrict foot traffic for at least four hours.

Cleanup … Immediately clean tools, equipment and overspray with soap and warm water.

PRECAUTIONS

KEEP FROM FREEZING. Do not apply when the temperature of the air and/or the concrete is less than 40° F (4° C). DO NOT MIX OR DILUTE WITH ANY OTHER PRODUCTS OR LIQUIDS. Do not use on surfaces that are later to be painted, tiled, hardened, sealed, or treated in any manner. Do not use on patios, sidewalks, or other areas where there is typically no wheel traffic to abrade the white film surface. Not recommended for use on residential applications.

HEALTH AND SAFETY

Direct contact may result in mild irritation. Read and follow all application, label, precautions, and health and safety information prior to use. Refer to Safety Data Sheet for complete health and safety information.

LEED INFORMATION

May help contribute to LEED credits:

- IEQ Credit 4.2: Low-Emitting Materials - Paints and Coatings
- MR Credit 2: Construction Waste Management
- MR Credit 5: Regional Materials

For most current data sheet, further LEED information, and SDS, visit www.wrmeadows.com.

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 08/16-3M

Appendix F: Interface Shear Calculations

Crack propagations along the UHPC-substrate interface were observed in some of the test slabs and therefore, calculations were performed to estimate the horizontal shear stresses at the interface at the time of failure and compare these values to the shear bond strength investigated in Task 2. A similar process was followed as discussed in detail by researchers at Iowa State University [41], following ACI 318 and using the following final equation:

$$V_h = \frac{V}{bd_e}$$

where V_h is the resultant horizontal shear stress, V is the vertical shear force (half of the measured P maximum load for each slab test), b is the width (21 inches for all slabs), and d_e is the distance between the centroid of the tension steel to the mid-thickness of the overlay (overall depth, minus the bottom clear cover, minus half the bar diameter, minus half the overlay thickness). The values used in the calculations are summarized in Table 21.

Table 21: Summary of Interface Shear Calculations

Slab	**V (kips)**	**b (in.)**	**d_e (in.)**	**V_h (psi)**
O-PR1	14.7	21	4.42	158
O-PR2	16.3	21	4.41	175
O-PW	11.5	21	3.80	143
O-NR	14.9	21	4.59	154

Overall, the maximum horizontal shear stresses at the interfaces ranged between 143-175 psi for all slabs. The average minimum bond shear stress for the Ductal UHPC was found to be 3.24 ksi and was discussed in detail in Task 2 Report. Therefore, the shear bond strength was on average 20.6 times that of the observed horizontal shear stresses at the interfaces during the slab tests. Similar to Iowa's findings [41], these results show that if the shear failures in the substrate concrete had not occurred, the UHPC overlay slabs could have resisted much higher loads before bond failure could have initiated at the interfaces.