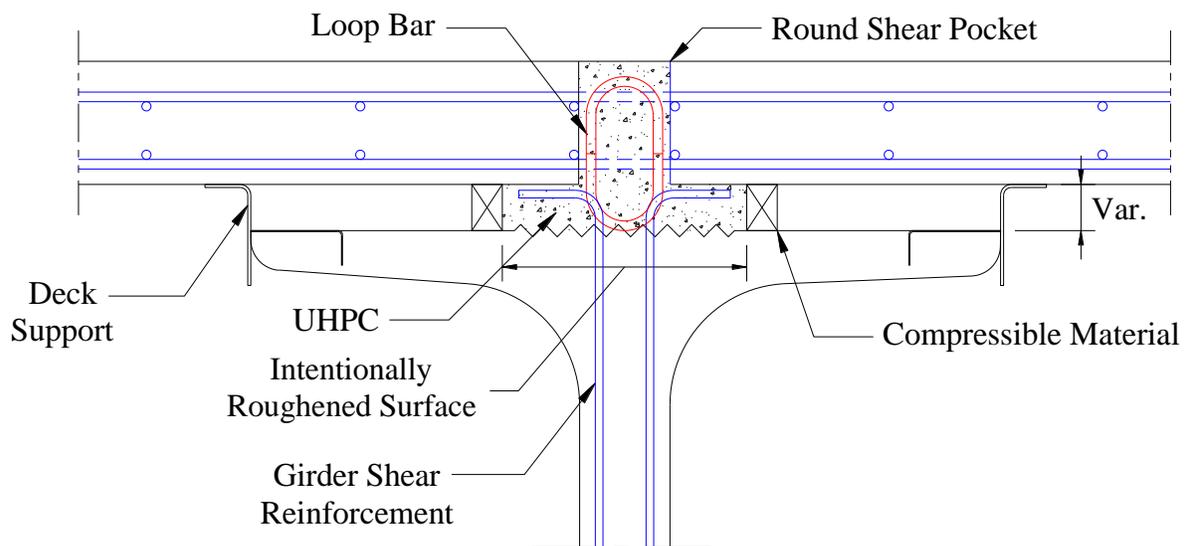


Precast Concrete Deck-to-Girder Connection using UHPC

Nebraska Department of Transportation (NDOT)

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Final Report



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Precast Concrete Deck-to-Girder Connection using UHPC

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ABSTRACT

The current method used in Nebraska for connecting precast concrete deck panels to precast/prestressed concrete girders to create composite section is extending shear connectors (i.e., threaded rods) from the concrete girder into HSS-formed shear pockets in the deck panels, and then filling the pockets and haunch area using self-consolidating concrete (SCC). This method requires high level of quality control/quality assurance (QA/QC) in spacing the shear connectors during girder fabrication as well as shear pockets during panel fabrication to avoid any conflict between them during erection. Recently, the implementation of ultra-high performance concrete (UHPC) in bridge connections is growing rapidly as an innovative method of accelerated bridge construction. A new precast deck-to-concrete girder connection is developed to eliminate any changes to girder design/production and any possible conflict between deck and girder reinforcement by utilizing UHPC and its exceptional mechanical properties, flowability, and workability. Typical girder shear reinforcement is used and positioned below the soffit of precast deck panels which eliminate the necessity of special shear connectors. The composite action between deck panels and girders is achieved through filling round shear pockets (4 - 8 in. in diameter) every 2 - 4 ft. along each girder line and haunch areas with UHPC. Loop bars are then embedded in the shear pockets while resting on the top flange of the girder to enhance the composite action with adequate development length.

The experimental investigation focused on the main two interface shear planes that control the structural performance of the new connection. The first interface plane is located at the soffit of deck panel at the shear pockets in monolithic UHPC and the second is at the top of concrete girder between the fresh UHPC and hardened conventional concrete (CC-UHPC). Direct shear, L-shape push-off, double shear push-off test tests were conducted to evaluate the interface shear resistance of monolithic UHPC. The interface shear resistance of CC-UHPC was evaluated through two different testing methods; slant shear and L-shape push-off tests. Then, full-scale push-off specimens were fabricated and tested to simulate the new connection and evaluate its structural performance and constructability. Based on the experimental investigation results, empirical equations are presented to predict the interface shear resistance of the two studied planes that are used to provide a design procedure for the new connection. Design and construction recommendations are concluded from the outcomes of experimental testing program.

The outcome of this study is an economical precast concrete deck-to-concrete girder connection that simplifies girder design and production as well as panel erection. The new connection provides full composite action while maintaining deck durability and speed of construction

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Chapter 1. Introduction

1.1. Background

In 2009, FHWA launched Every Day Counts (EDC) program to speed up highway construction. Accelerated Bridge Construction (ABC) is bridge construction that uses innovative planning, design, materials, and construction methods in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges (FHWA, 2011). FHWA works with states to identify and implement innovations for ABC, such as:

- Geosynthetic Reinforced Soil-Integrated Bridge System (GRS-IBS)
- **Prefabricated Bridge Elements and Systems (PBES)**
- Slide-in Bridge Construction (SIBC)
- **Ultra-High Performance Concrete (UHPC) for Connections**

FHWA published national bridge inventory for bridges across US in 2016. One of the classifications is bridge deck structure type as shown Figure 1.1. Cast-in-place (CIP) concrete deck is the most common used bridge deck type which represents 59.3% of the total bridge deck systems. The CIP deck system requires a long duration for forming deck, placing reinforcement, casting and curing concrete which leads to long traffic lane closures and detouring. Also, the inconsistent quality is a major challenge facing CIP concrete which caused by several factors such as environmental conditions and placing, finishing, and curing of concrete. As a result, CIP concrete decks experience excessive early-age shrinkage cracking which decreases bridge durability and requires overlay. These disadvantages highly impact the construction time and project budget.

One of ABC innovations is the implementation of prefabricate bridge elements in construction. Recently, precast concrete deck panels have been successfully used in ABC projects in various forms and systems. Casting deck panels out-side the construction site in a high-quality controlled environment reduces the deck cracking and provides more durable elements. Then, the precast deck panels are mobilized to the construction site which reduce the construction time and traffic closure. These advantages make the precast concrete deck panels system more durable and cost-effective compared to CIP concrete deck system. The precast deck panels are connected to the supporting girders through longitudinal and transverse connections or/and shear pockets filled with flowable grouting material.

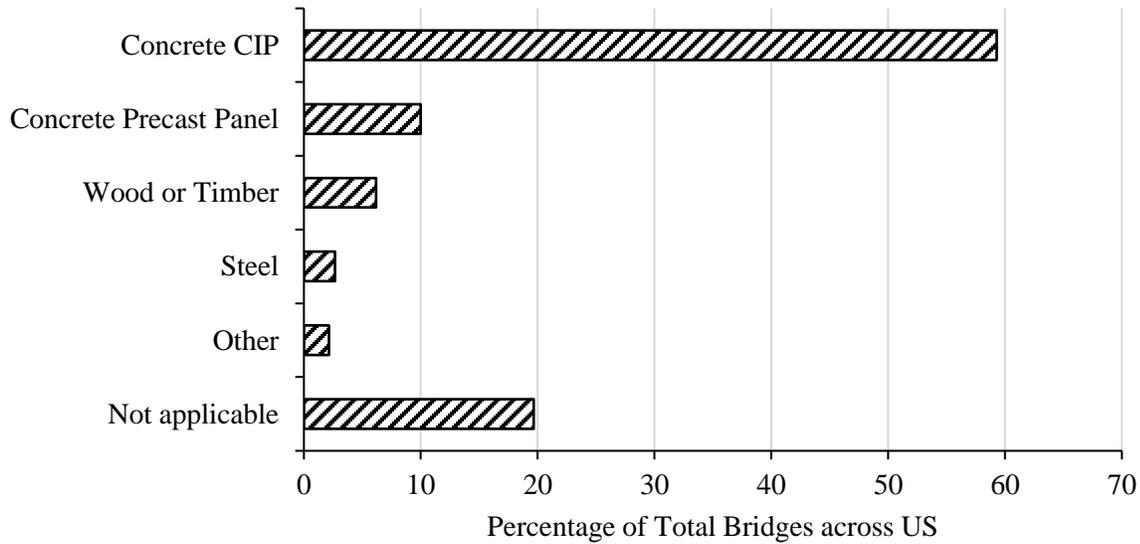


Figure 1.1: National Bridge Inventory by Deck Structure Type in 2016.

Another ABC innovation is using Ultra-High Performance Concrete (UHPC) in bridge connections. UHPC is a new generation of cementitious materials that has exceptional mechanical properties, durability, and workability. The low water-to-binder ratio, high binder content, use of supplemental cementitious material, high particle-packing density, and use of steel fibers significantly enhance the fresh and hardened UHPC properties compared to conventional concrete (CC). According to ASTM C1856-17, UHPC is characterized by a minimum specified compressive strength of 17 ksi, maximum aggregate size less than 1/4 in. and flow between 8-10 in. UHPC became commercially available in the U.S. through several proprietary sources around the year 2000. Since its introduction to the commercial market, the use of UHPC in various applications has been the focus of multiple research endeavors. The exceptional durability and mechanical properties make UHPC an ideal grouting material for field casting connections that reduce cracks and leakage in bridge connections.

1.2. Problem Statement

The method used in Nebraska for connecting precast concrete deck panels to concrete girders to create composite section is extending shear connectors (i.e., threaded rods) from the concrete girder into HSS-formed shear pockets in the deck panels, and then filling the pockets and haunch area using self-consolidating concrete (SCC) as shown in Figure 1.2. This method requires high level of quality control/quality assurance (QA/QC) in spacing the shear connectors during girder fabrication as well as shear pockets during panel fabrication to avoid any conflict between them during erection. It also requires shear connectors to achieve minimum embedment in the shear pockets to develop the design capacity, which necessitates adjusting connector's height to compensate for girder camber. Therefore, there is a need

for research to investigate alternative economical deck-to-girder connection that simplifies girder design and production as well as panel erection. The new connection provides full composite action while maintaining deck durability and speed of construction.

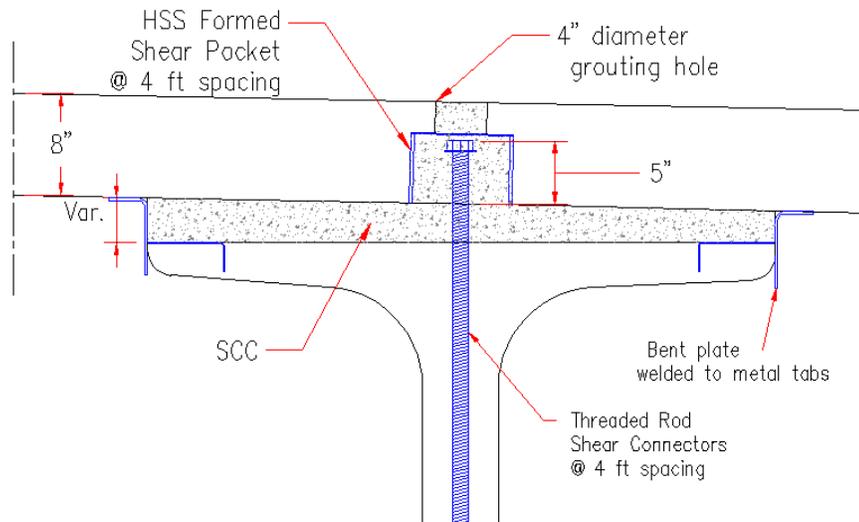


Figure 1.02: Precast Concrete Deck-to-Girder Connection using threaded rods and HSS-formed shear pockets in the deck panels.

1.3. Research Objectives

The general objective of this research is to promote the use of UHPC in the construction of precast deck-to-girder system bridges. The specific objectives are to:

1. Develop a new UHPC connection between precast concrete deck panels and precast/prestressed concrete girders that eliminates any changes to girder design/production and any possible conflict between deck and girder reinforcement.
2. Investigate the interface shear resistance of monolithic UHPC.
3. Investigate the interface shear resistance of fresh UHPC cast on hardened conventional concrete.
4. Investigate the constructability and structural performance of the new connection through full-scale push-off test.
5. Provide design procedure and recommendations.

1.4. Report Outline

This report consists of six chapters as follows.

Chapter 1 - Introduction

Chapter 2 - Literature Review: The literature review presents the existing deck-to-girder connections using UHPC and the interface shear resistance of UHPC. Two different interface shear planes are controlling the design of deck-to-girder bridge connections; interface shear plane in monolithic UHPC and between fresh UHPC and hardened conventional concrete (CC-UHPC). This chapter summarizes the different test methods conducted to evaluate the resistance of these two planes

Chapter 3 – Proposed Deck-To-Girder Connection: This chapter introduces a new UHPC connection between precast concrete deck panels and precast/prestressed concrete girders that eliminates any changes to girder design/production and any possible conflict between deck and girder reinforcement. The new connection makes advantage of the excellent mechanical properties of UHPC as well as its exceptional workability and durability. UHPC is used to fill the shear pockets and haunches between deck panels and supporting girders to create composite systems. This connection does not require girder shear connectors to be embedded inside deck panels, which eliminates any possible conflicts during panel erection. Also, the construction sequence of the new bridge connection and study methodology are presented.

Chapter 4 – Experimental Investigation: This chapter illustrates the experimental investigation procedure, small-scale and full-scale testing, to evaluate the interface shear resistance of monolithic UHPC and of fresh UHPC cast on hardened conventional concrete (CC-UHPC). Direct shear, L-shape push-off, double shear push-off tests were conducted to evaluate interface shear resistance of monolithic UHPC. The literature review conducted on interface shear resistance of CC-UHPC was summarized and analyzed to propose prediction equations. Then, slant shear test and L-shape push-off test were conducted to evaluate and validate these equations. The constructability and structural performance of the proposed connection was investigated through full-scale push-off tests. The full-scale specimens were designed to use 3 ft. spacing between shear pockets.

Chapter 5 – Design Procedure: This chapter provides a design methodology for the proposed connection based on the prediction equations obtained from experimental investigations. An example bridge from PCI Bridge Design Manual 2014 (PCI BDM Ex. 9.1a) is used to present the design procedure of the proposed connection. Design aid charts were generated to simplify the connection design.

Chapter 6 – Conclusions and Recommendations: This chapter presents a brief summary and conclusions drawn from the experimental investigation and construction recommendations for using UHPC as a grouting material in a new deck-to-concrete girder concrete connection. These recommendations were

developed based on the test results and the experience gained from the full-scale push-off testing in Chapter 4.

Chapter 2. Literature Review

2.1. Introduction

The literature review presents the existing deck-to-girder connections using UHPC and the interface shear resistance of UHPC. Two different interface shear planes are controlling the design of deck-to-girder bridge connections; interface shear plane in monolithic UHPC and between fresh UHPC and hardened conventional concrete (CC-UHPC). This chapter summarizes the different test methods conducted to evaluate the resistance of these two planes

2.2. Deck-To-Girder Bridge Connection Using UHPC

This section summarizes the literature review conducted on UHPC used for deck-to-girder connections. Typically, this connection is made of shear connectors, such as bent rebars or threaded rods in concrete girders, and shear studs in steel girders, that are embedded into discrete shear pockets or continuous troughs in the precast concrete deck panels. Then, a flowable concrete or grout is used to fill these pockets or troughs to establish the connection. One of the disadvantages of these systems is that shear connectors are required to have minimum embedment into the shear pockets/troughs to develop the design capacity, which necessitates high level of QA/QC and complicates girder and panel production.

A new UHPC connection concept was developed to eliminate this problem and simplify production and erection procedure, which consequently improve construction speed and economy. A series of interstate highway bridges near Syracuse, NY were constructed using this UHPC deck-to-girder connections developed by NYSDOT (Graybeal 2014). The connection consists of panel-to-panel longitudinal shear key with lap spliced transverse reinforcing rebars over the girder lines. Conventional shear connectors ($\frac{3}{4}$ in. x $3\frac{1}{4}$ in.) are welded to the top flange of the steel I-girder as shown in Figure 2.1. The V-shaped shear keys have roughened/exposed aggregate finish to properly bond with the field-cast UHPC that connects the adjacent panels to each other and to the supporting girders. Dimensions of the longitudinal joint is typically 7 in. at the top and bottom of the deck slab and 10 in. at the middle of deck slab. Length and spacing of lap splices depend on bar size and type.

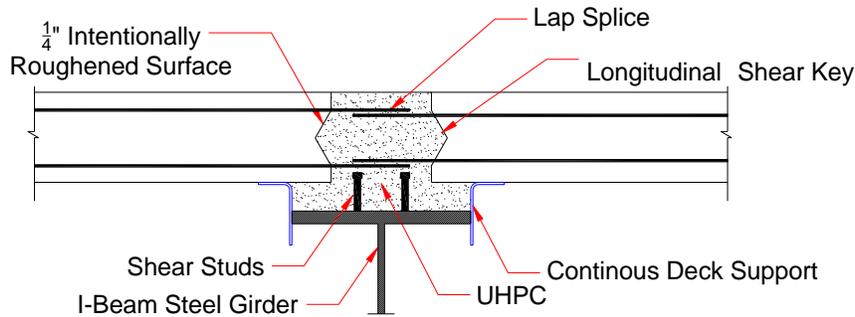


Figure 2.1: Panel-to-Panel Connection over Steel Girder (Graybeal 2014)

Another precast concrete deck-to-steel girder connection was recently developed and tested using UHPC (Graybeal 2014). In this connection, $\frac{3}{4}$ in. x $3\frac{1}{4}$ in. shear studs are installed on the girder top flange similar to cast-in-place (CIP) deck construction (i.e. similar spacing requirements). A 10.5 in. wide and 4.5 in. deep trough with exposed aggregate finish is formed in precast concrete deck slab with 2 in. grouting holes every 24 in. over each girder line as shown in Figure 2.2a. Shear studs are kept below the bottom mat of deck reinforcement without embedment in the deck panels to simplify panel and girder production and eliminate any conflicts during panel installation. An interstate highway bridge near Syracuse, NY was constructed using this connection concept with single field casting of UHPC through grouting holes for each girder line to hide the connection and eliminate the need for deck overlay. The same concept can be used with concrete girders by replacing the shear studs with conventional shear reinforcement (i.e. U bars) that are extended above the top flange and below the bottom mat of deck reinforcement (Graybeal 2014) as shown in Figure 2.2b. This connection has been tested but not implemented yet.

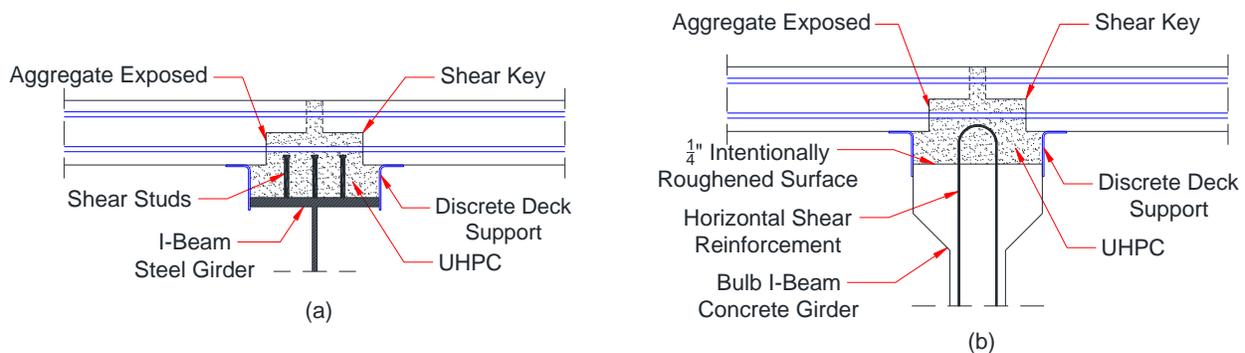


Figure 2.2: Hidden UHPC Deck-to-Girder Connection in Steel Girder (a) and Concrete Girder (b) (Graybeal 2014)

Recently, a study was conducted on implementing UHPC as a grout for deck-to-steel girder composite connection using two new concepts (Haber et al. 2017): a) using shear lugs through deck slab with different areas, and b) using vertical rebar dowels from the deck slab to connection without lugs as

shown in Figure 2.3. The second concept was investigated for different haunch thicknesses 5 in. and 3.5 in. and different distributions of shear studs. Push-off specimens were fabricated by having a symmetric layout with W10x60 steel beam at the middle connected to two 20 in. x 24 in. precast slabs through a grouted UHPC connection. The push-off test was performed by applying the shear force on the steel stub and evaluate the connection performance at the shear interface surface. The UHPC shear lugs have shown to be effective in transferring shear forces and the location and number of shear studs have an effect of the capacity of the connection. Adding rebar dowels to the connection increases the shear resistance, develops better anchorage, and achieve ductile failure behavior due to rebar dowel action.

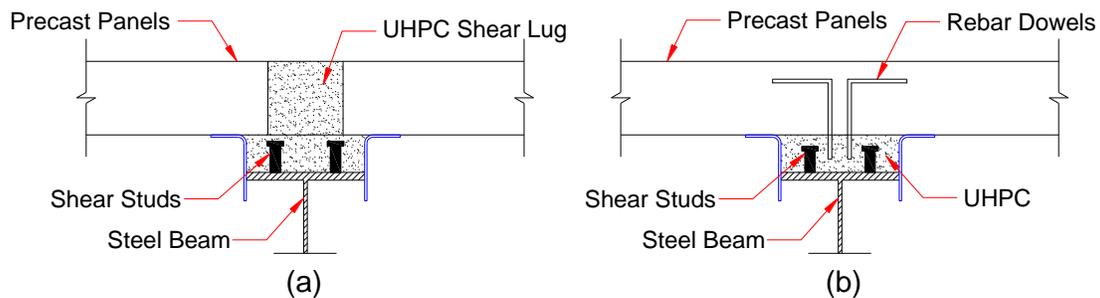


Figure 2.3: New Haunch-to-Deck Connection Using UHPC through Shear Lug (a) and Rebar Dowels (b) (Haber et al. 2017)

2.3. Interface Shear Resistance of UHPC

The interface shear resistance is the maximum shear stress that prevents the relative slide between two different concrete layers. The interface shear behavior between two different concretes was first presented by Birkeland and Birkeland (1966). Shear friction, concrete surface cohesion and dowel action are the main mechanisms that control the interface shear resistance as shown in Figure 2.4. The interface shear resistance of concrete-to-concrete planes controls the design of many reinforced concrete structures. The capacity of the horizontal shear planes governs the achievement of composite action between precast concrete girders and bridge deck. In recent years, the use of UHPC in bridge construction has been growing significantly. The exceptional properties of UHPC simplify the deck-to-girder connections in Accelerated Bridge Construction (ABC) projects. The interface shear resistance of UHPC can be summarized in four categories with and without reinforcement:

1. **Interface shear resistance of Monolithic UHPC**
2. **Interface shear resistance between hardened conventional concrete and fresh UHPC (CC-UHPC)**
3. Interface shear resistance between hardened UHPC and fresh conventional concrete (UHPC-CC)
4. Interface shear resistance between hardened and fresh UHPC (UHPC-UHPC)

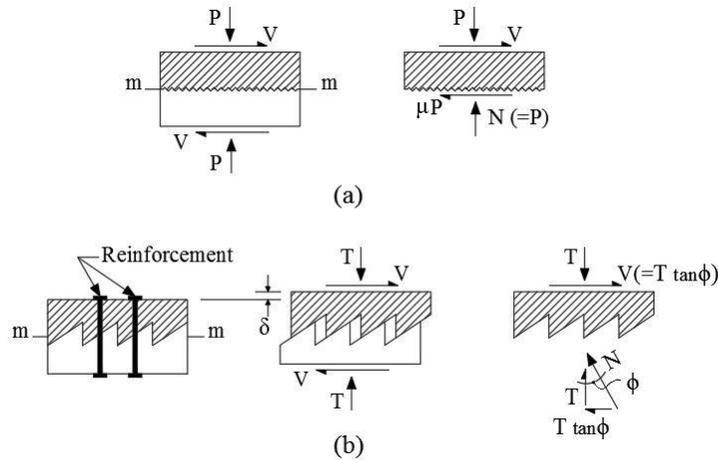


Figure 2.4: Interface Shear Friction Theory (Brikelan and Birkeland 1966).

2.3.1. Interface Shear Resistance of Monolithic UHPC

Crane (2010) performed vertical interface shear push-off tests of monolithic UHPC specimens to determine whether ACI 318 (2008) and AASHTO LRFD (2007) equations of interface shear are applicable to monolithic UHPC. UHPC specimens with un-cracked and pre-cracked interfaces, and with reinforcement ratios of 0 and 0.5% were tested. Three identical push-off specimens were tested for each combination of interface type and reinforcement ratio as shown in Figure 2.5. Test results indicated that the ultimate interface shear strength was significantly higher than that predicted for monolithic concrete in all cases. Regression analysis was performed to estimate UHPC cohesion and friction coefficients (c and μ). For un-cracked UHPC, $\mu = 4.5$, and $c = 2$ ksi were proposed, and for cracked monolithic UHPC, $\mu = 4.0$, and $c = 0.65$ ksi were proposed. These high values were attributed to the contribution of the steel fibers distributed across pre-existing cracks even when no mild shear reinforcement is used. Also as expected, the specimens with reinforced UHPC exhibited more ductile behavior than those with unreinforced UHPC. The average interface shear strength of un-cracked monolithic UHPC increases by 48% with the increase of interface reinforcement ratio from 0 to 0.5% as shown in Figure 2.6.

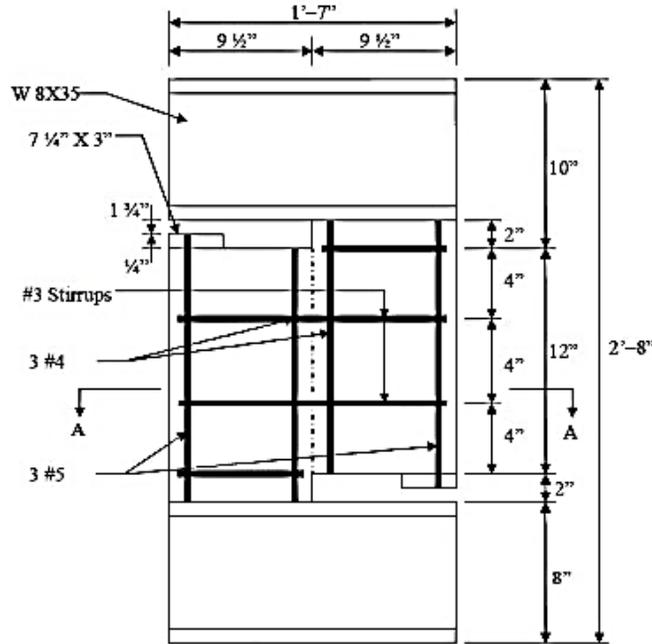


Figure 2.5: Vertical Interface Shear Push-off specimen of Monolithic UHPC (Crane 2010)

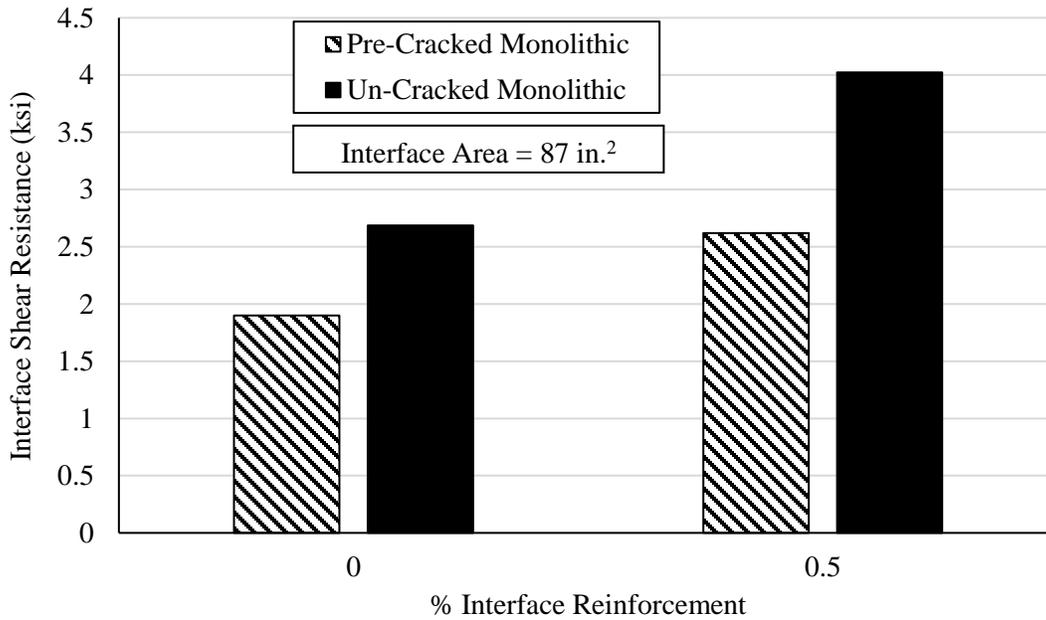


Figure 2.6: Average Interface Shear Strength of Monolithic UHPC with and without Interface reinforcement (Crane 2010)

Maroliya (2012) investigated the behavior of reactive powder concrete (which is another term for UHPC) in direct interface shear. A series of direct shear specimens having inverted “L” shape in shear failure plane were tested using monolithic UHPC with different percentages of steel fibers as shown in Figure 2.7. Test results showed that plain UHPC samples failed in a brittle manner at the first-crack load,

which happens to be the maximum load taken by the specimen. On the other hand, samples having 2.5% fibers indicated multiple visible cracks, while samples having 2% fibers resulted in a maximum load much higher than the first-crack load, which clearly reflects failure after the strain hardening of the material. These results helped concluding that UHPC exhibits a ductile failure mode depending on the percentage of fibers. Figure 2.8 shows the effect of different fiber content and curing methods on the direct shear strength of monolithic UHPC. Results also indicated an average value of direct shear strength for normal cured monolithic UHPC with 2% fiber volume fraction of about 2 ksi.

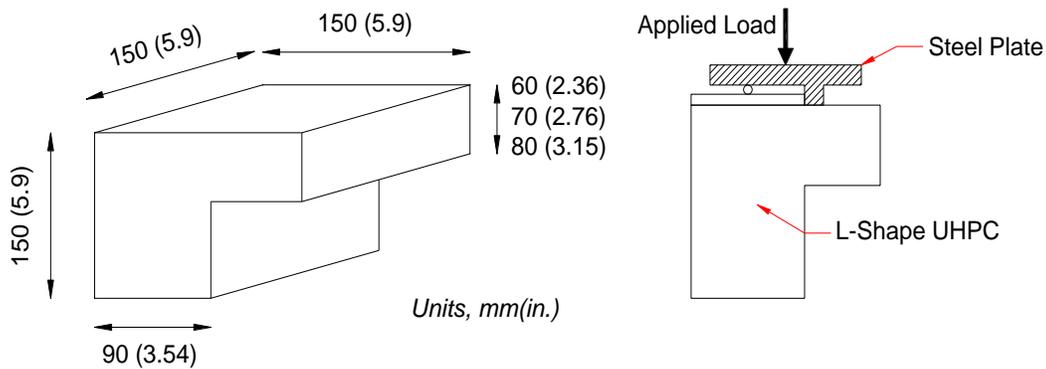


Figure 2.7: Shear Testing on Inverted L-Shape UHPC Specimen (Maroliya 2012)

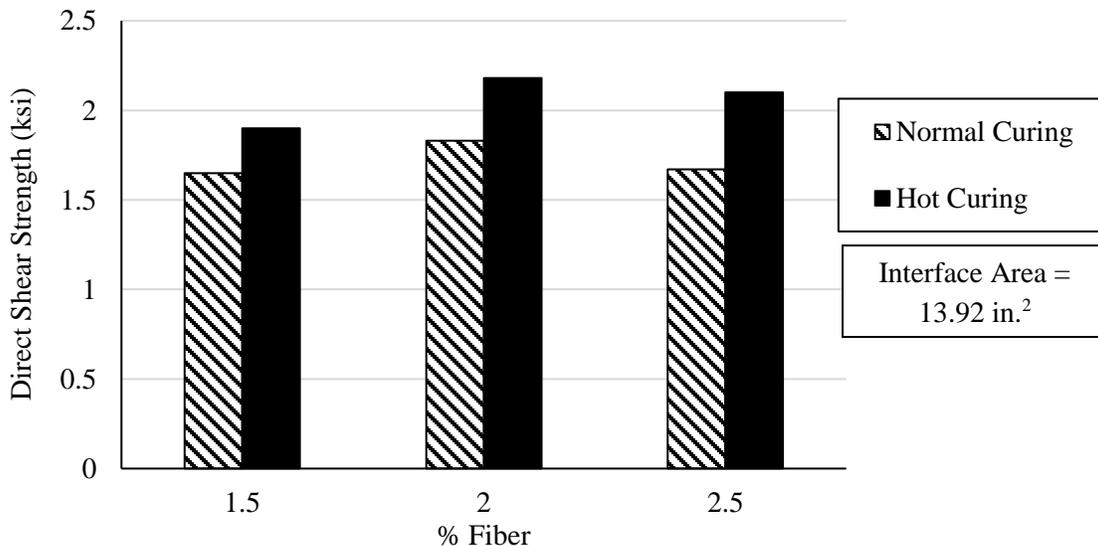


Figure 2.8: Effect of Fiber Content and Curing Methods on Direct Shear Strength of Monolithic UHPC without Interface reinforcement (Maroliya 2012)

Jang et al. (2017) conducted vertical shear test on L-shape specimen to evaluate the monolithic interface shear resistance of UHPC without interface reinforcement as shown in Figure 2.9. The UHPC matrix consists of water-to-binder ratio (W/B) of 0.14, type I/II Portland cement, Australian silica sand, and

silica with a fiber content of 1.5% of volume. The UHPC achieved 29.08 ksi at 91 days. A vertical load was applied on the specimen with a rate of 0.024 in./min. till failure. Four LVDTs were used to capture the horizontal and relative vertical displacement in the L-shape specimen. The interface shear resistance of monolithic UHPC without interface reinforcement was 2.72 ksi with interface shear area of 46.50 in.².

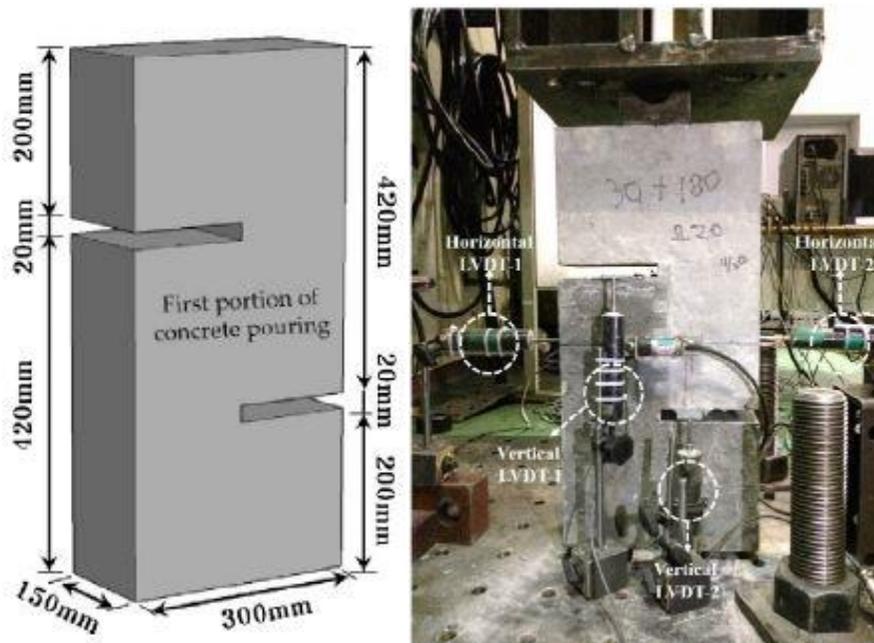


Figure 2.9: Monolithic L-Shape UHPC Specimen Test Setup and Specimen Dimensions (Jang et al. 2017)

Small and large scale push off testing were performed to obtain the direct shear capacity of UHPC (Haber et al. 2017). Three small specimens composed of 6 in. long with 2 in. square cross section beams were tested for a given age by applying vertical load on the beam that was fixed by square supports from both ends. The UHPC were poured from one end for controlling the fiber orientation to be perpendicular to the applied loads. 14 in. by 24 in. two precast concrete slab with lug pockets were pre-fabricated and the lugs were filled with UHPC with a stub. A vertical load was applied on the UHPC stub to investigate the shear capacity of the proposed UHPC shear lugs. The direct shear testing for the small and large specimens are shown in Figure 2.10. The Specimens exhibit a double shear failure and the test results are summarized in Figure 2.11. A range of 4 ksi to 8 ksi UHPC direct shear capacity could be achieved according to the tested specimens.

Slant shear test is conducted to evaluate the bond resistance over the interface plane between two different types of materials. The type and dimensions of slant shear specimens and interface angle changes according to applied codes as shown in Table 2.1. However, British and French standards use prism specimen and ASTM C882 uses cylindrical specimen, the interface plane angle with the horizontal axis is 60°. Several parameters influence slant shear bond capacity such as interface plane angle and surface texture.

Table 2.1: Slant Shear Composite Specimen Dimensions in Different Codes

Code	Type of Specimen	Dimensions	Interface Plane Angle with Horizontal Axis
ASTM C882/C882M-13a	Cylinder	3x6 in.	60°
BS EN 12615:1999	Prism	3.9x3.9x15.7 in. or 1.6x1.6x6.3 in	60°
French standard NFP 18-872	Prism	3.9x3.9x11.8 in	60°

ASTM C882/C882M-13a is mainly for determining the bond resistance of adding a layer of epoxy-resin-base material between either two hardened or hardened and fresh Portland-cement concrete. The slant shear test is performed on 3 in. by 6 in. specimens with an interface plane angle of 60° with horizontal axis as shown in Figure 2.12. The specimen sections are prepared by placing Portland-cement mortar in the mold in two layers of approximately equal volume which was uniformly rodded 25 time per each layer. The compressive strength of the concrete section should have at least 4500 psi at 28 days after being cured. Based on 60° angle inclined interface plane, the area of the elliptical interface plane is twice the area of the specimen base. The specimens shall be tested at 73 ± 2 °F in compression after capping in accordance with test method C39/C39M. A minimum of three composite specimens are required for each testing type.

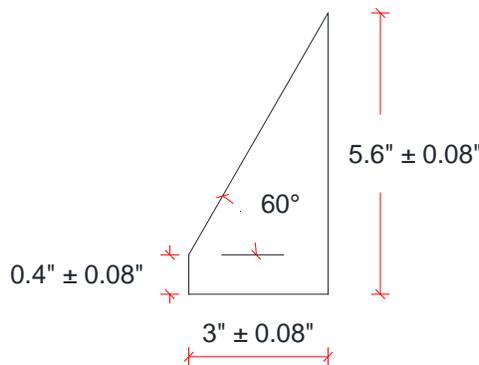


Figure 2.12: Portland-Cement Concrete Section Dimensions.

The interface shear resistance between a UHPC overlay and hardened normal concrete substrate with different textures was investigated by three different test procedures; slant shear test, flexural test, and split prism test (Harris et al. 2011). The slant shear test was performed according to ASTM C882/C882M to evaluate the interface shear resistance using 3×6 in. composite cylinders. A total of twenty-seven composite cylinders were fabricated and tested. The hardened section composed of Type III normal concrete mortar that had 5000 psi at 28 days with moist curing. Three different surface textures were applied to the interface shear plane; smooth (no surface preparation), low roughened (average depth of 0.1 in.), and high roughened (0.20 in. transverse grooves) surfaces as shown in Figure 2.13(a). Wire brush treatment and handheld metal grinder were used to obtain the low and high roughened surfaces respectively. Also, trapezoidal shear key (fluted), with 0.50 in. depth and 0.63 in.² area, was prepared as a precast scenario for using UHPC as a protective overlay. The hardened concrete mortar sections were placed back inside the molds and filled with UHPC. The composite specimens were cured under ambient conditions for 10 days till the UHPC and normal concrete gained compressive strength of 15 ksi and 5 ksi, respectively. The composite specimens were loaded under compression until failure happened either on the interface plane or the concrete crashed as shown in Figure 2.13(b). The interface shear resistance was calculated by dividing the peak load by the interface surface area.



(a)



(b)

Figure 2.13: Slant Shear Test; (a) Mortar Different Roughened Surfaces and Trapezoidal Shear Key, (b) Test Setup (Harris et al. 2011)

However, the composite specimens with smooth interface exhibited failure along interface plane, the roughened interface specimens had a normal concrete failure under compression as shown in Figure 2.14. The average interface shear resistance for smooth surface was 1.6 ksi and it increased with 28%, 56%, and 57% with applying low roughened, high roughened surfaces, and shear key respectively as shown in

Figure 2.15. The slant shear test results show that the interface shear resistance increases with higher interface roughening.



Figure 2.14: Failure Modes; (a) Failure along Interface Plane, (b) Normal Concrete Failure (Harris et al. 2011)

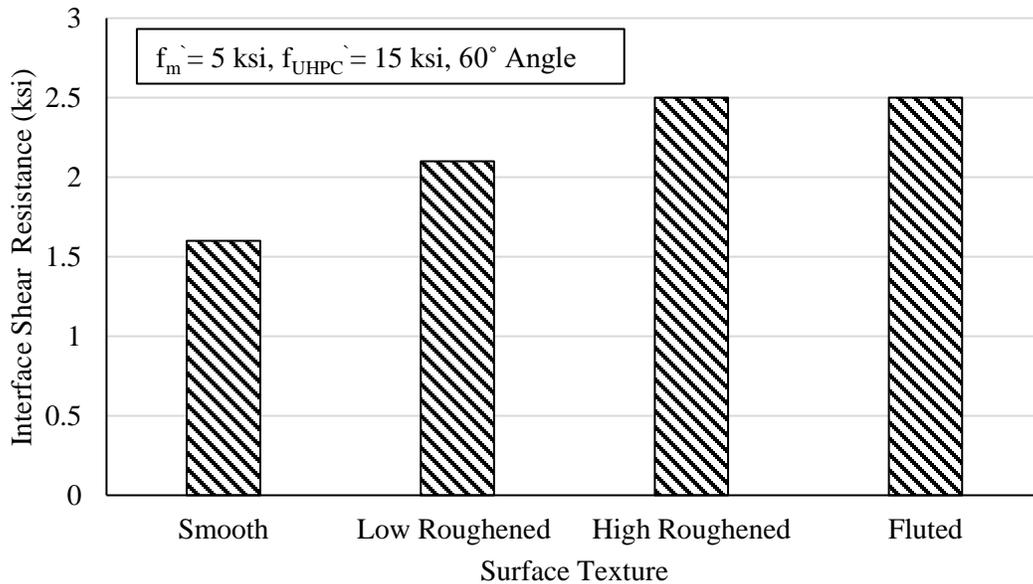


Figure 2.15: Interface Shear Resistance of Cement Type III Mortar with Different Surface Textures (Harris et al. 2011)

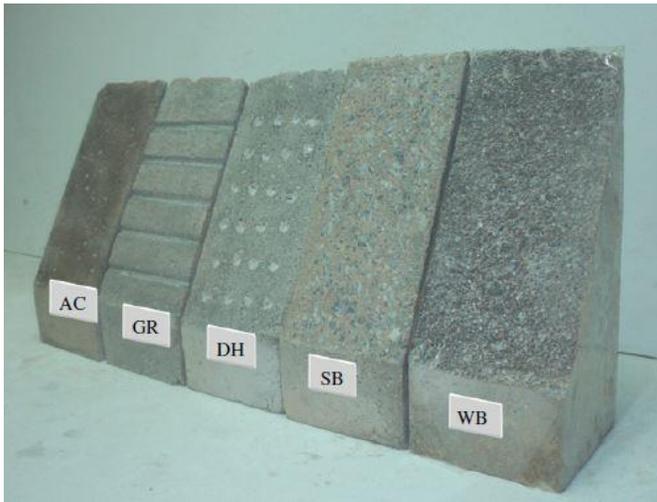
Tayeh et al. (2012) investigated the mechanical and permeability properties of interface between CC substrate which represents old concrete structures and an overlay of ultra-high performance fiber concrete (UHPFC) as a repair material. The interface shear resistance and influence of different surface roughening were evaluated through performing slant shear test and splitting tensile test. The mix proportions of UHPFC and CC are shown in Figure 2.16(a). The slant shear composite specimens were fabricated using prism of 3.9x3.9x11.8 in. with interface angle with vertical of 30°. the interface plane was prepared with five different surface textures; without preparation, sand blasted, wire brushed, drilled holes (0.4 in. diameter and 0.2 depth), and grooved (0.4 in. width and 0.2 in. depth) as shown in Figure 2.16(b).

The compressive strength of CC and UHPFC at 28 days are 6.53 and 24.66 ksi, respectively. The test was conducted according to ASTM C288 and the test setup is shown in Figure 2.16(c).

Table 1
Mix proportions for NC substrate and UHPFC.

Concrete type (kg/m ³)	NC substrate	UHPFC
OPC (Type 1, 42.5R)	400	768
Coarse aggregate (max. 12.5 mm)	930	–
River sand (F.M. = 2.4)	873	–
Mining sand (<1180 μm)	–	1140
Silica fume (23.7 m ² /g)	–	192
Steel fiber ($L_f = 10$ mm, $d_f = 0.2$ mm)	–	157
Superplasticizer (PCE-based)	4	40
Water	200	144
Total	2407	2441
W/B	0.5	0.15
Cube strength, $f_{c,28d}$	45 MPa	170 MPa
Split cylinder tension strength, $f_{sp,28d}$	2.75 MPa	15.3 MPa

(a)



(b)



(c)

Figure 2.16: (a) Mix Proportions of UHPFC and CC, (b) Surface Textures, and (c) Test Configuration (Tayeh et al. 2012)

Four different failure modes were observed; pure interfacial failure, interfacial failure with minor CC cracking, interfacial failure with CC fracture, and substrata failure. The interface shear resistance was calculated by dividing the maximum applied load by the interface contact area. The sand-blasted texture specimens give the highest interface shear resistance of 2.58 ksi. The surface texture clearly influences the interface shear resistance, as compared to surface without preparation, the interface shear resistance

increases with 105%, 60%, 47%, and 41% for sand blasted, grooved, wire brushed, and drilled holes surfaces as shown in Figure 2.17.

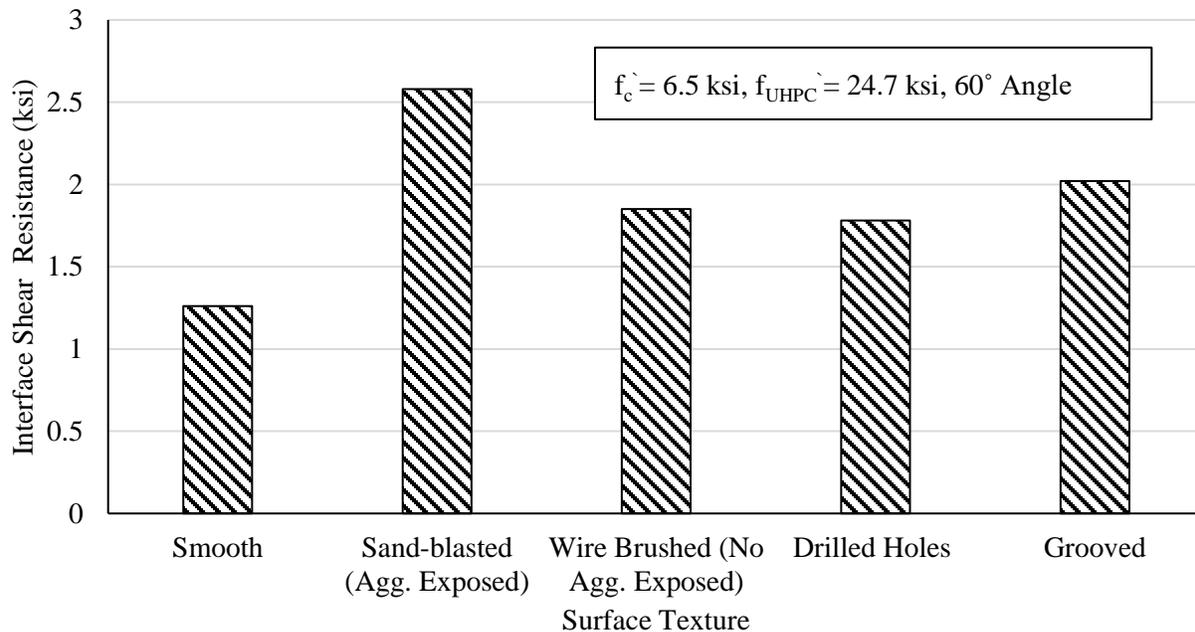


Figure 2.17: Different Surface Texture Effect on Interface Shear Resistance of CC-UHPC (Tayeh et al. 2012)

Muñoz (2012) conducted a study on using UHPC as a repair material by investigating the interface shear resistance between UHPC and normal concrete. The interface shear resistance was evaluated by three different test methods; slant shear test, splitting prism test, and pull-off test. The slant shear test was conducted to obtain the interface shear resistance of different surface preparation treatment and interface angles. The slant shear composite specimens were 3.5x3.5x14 in prism to allow casting concrete substrate contrasting ASTM C 882 that use mortar substrate as shown in Figure 2.18. This study focused on four different surface textures; roughening, brushed, sandblasted, grooved, and roughened (exposed aggregate), and two different interface angles with horizontal axis, 60° and 70° . The normal concrete sections were casted in wooden forms and cured in two stages; 24 hours in moist cure before demolding and Then in a lime water tank for 28 days. The compressive strength of normal concrete mixes was 6.46 ksi, 6.61 ksi, and 8.11 ksi for grooved and brushed, roughened, and sandblasted surface texture specimens respectively. A steel brush and drill-bit, sandblasting equipment, wet saw, and concrete retarder were used to obtain the brushed, sandblasted, grooved, and roughened interface surface textures respectively. Two different methods were used to evaluate the roughening degree, the macrotexture depth test and the Concrete Surface Preparation index (CSP) given by ICRI guide. Figure 2.19 and Table 2.2 shows the different surface textures and the degree of roughening measurement.

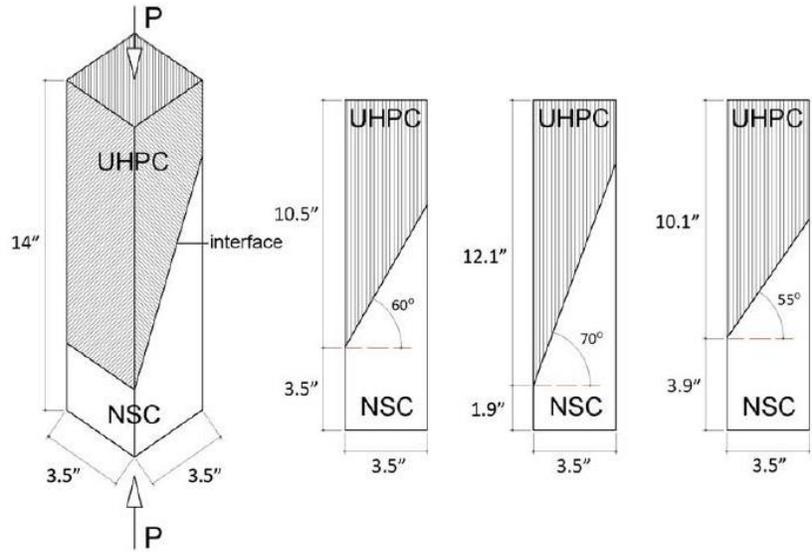


Figure 2.18: Slant Shear Composite Specimen Dimensions (Muñoz 2012).

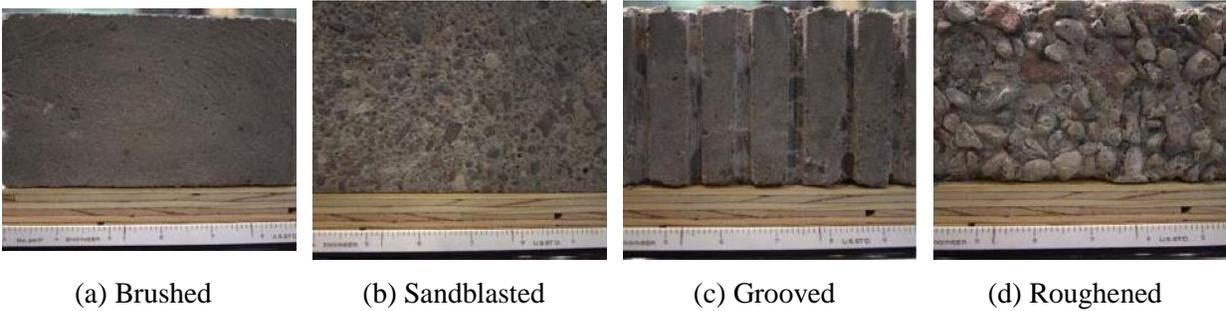


Figure 2.19: Different Surface Textures (Muñoz 2012).

Table 2.22: the macrotexture depths of prepared surfaces ((Muñoz 2012)

Surface	ICRI Profile	Macrotexture Depth (in)
Brushed	1,3	0.03
Sandblasted	4,5	0.03
Grooved	Not applicable	Not applicable
Rough	Aggregate exposure >8,9	0.09

The composite specimens consisted of hardened CC blocks with prepared interface surface texture after curing in a water tank and Ductal®JS1000 UHPC poured on the blocks. Four composite specimens were tested at 8 days for each texture. A load rate of 35 psi/second was used to apply compression load using Baldwin CT 300 hydraulic load frame till failure as shown in Figure 2.20. The tested specimens exhibited different failure modes. The slant shear specimens with 60° interface angle and 8 days of UHPC exhibited CC failure. However, the 70° interface angle brushed surface specimens had bond failure, the other surface textures expressed CC failure. Figure 2.21 shows the effect of interface angle on the interface shear strength for different surface texture at 8 days of UHPC. The interface Shear Resistance was calculated by dividing the maximum applied load by the interface contact area. The interface shear

resistance of sandblasted specimens is the highest compared to the other surface texture specimens. The higher compressive strength of sandblasted CC section might give a wrong conclusion as mention by the authors. Failure modes and interface shear resistance are affected by the change of the interface angles. The interface shear resistance at 8 days for all surface preparations exceeded the requirements specified by ACI 546.3R-06 at 7 days and satisfied the minimum bond requirements for 28 days.



Figure 2.20: Slant Shear Test Configuration (Muñoz 2012).

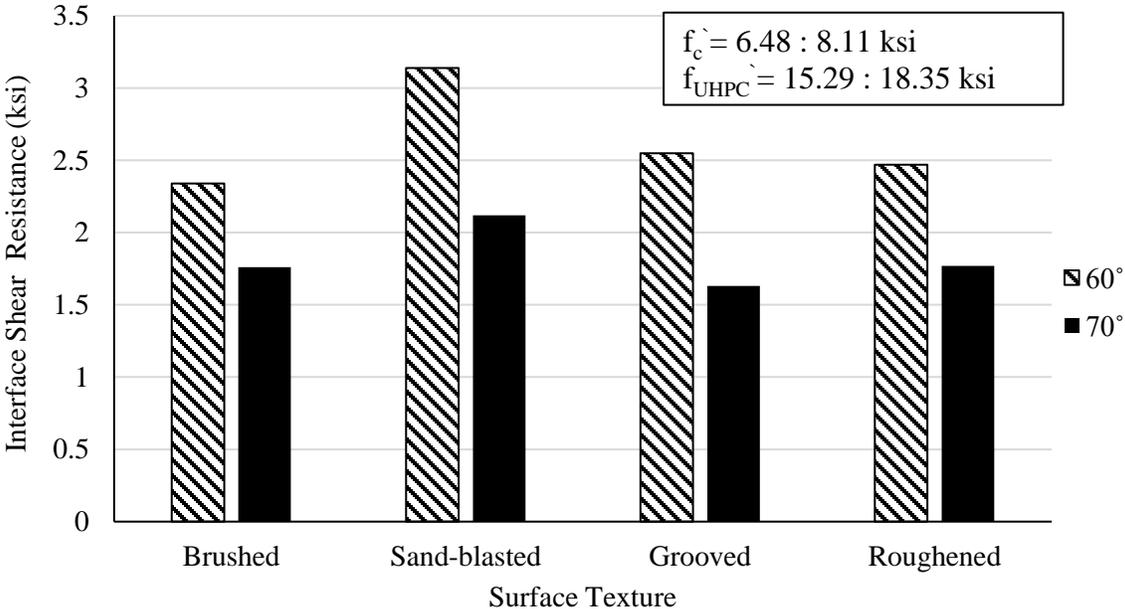


Figure 2.21: Effect of Interface Angle on Interface Shear Resistance at 8 Days of UHPC (Muñoz 2012).

Rangaraju et al. (2013) performed a study on developing local UHPC using available materials in South Carolina and evaluating its performance as shear key grout for NEXT-D bridge design. Slant shear test was conducted to evaluate the interface shear resistance of the local UHPC as a part of determining the

local UHPC properties. The slant test was performed according to ASTM C882 with modifications, using normal concrete representing bridge deck instead of concrete mortar. The normal concrete was casted in 3x6 in cylinders and moist-cured for a 28-day period. The range of normal concrete compressive strength was from 5.92 to 7.61 ksi. The interface surface was then treated by sand-blasting to obtain a roughened surface. Four different UHPC mixes were poured on the top of normal concrete section, demolded after 1 day, and moist-cured for 6 days. The composite specimens were tested at 7 and 28 days under compression rate according to ASTM C39. However, most of the specimens failed in the normal concrete portion, one specimen exhibited bond failure. The interface shear resistance and failure modes are shown in Table 2.3.

Table 2.3: Different Mix Proportions Used in Evaluating Local UHPC Properties, lb/Yard³ (Rangaraju et al. 2013)

UHPC ID	Cement	Sand	Silica fume (SF)	Water	SP, %	Steel microfiber**
UHPC 1	1601	2002	-	320	RQ	-
UHPC 2	1300	1949	260	312	RQ	-
UHPC 3	1273	1909	255	305	RQ	270
UHPC 4	1249	1873	250	300	RQ	270

*SP quantity is expressed in terms of percentage by weight of the total cementitious material (cement + silica fume)

**microfiber dosage is expressed in terms of percentage by volume of the non-microfiber mixture
RQ indicates required quantity to obtain a full flow of 150%

Table 2.4: Slant Shear Test Results and Failure Modes (Rangaraju et al. 2013)

UHPC ID	Maximum Applied Force					
	7-days			28-days		
	Average, kips	COV, %	Failure Location	Average, kips	COV, %	Failure Location
UHPC 1	28.3	2.0	Concrete	46.2	4.7	Concrete
UHPC 2	32.7	8.4	Concrete	56.2	5.2	Concrete+UHPC
UHPC 3	64.6	6.6	Bond+Concrete	58.5	9.6	Concrete
UHPC 4	62.7	11.2	Concrete	66.9	10.5	Concrete

The shear transfer behavior across the interface plane between UHPC and CC was investigated analytically and experimentally by conducting slant shear test and flexural test (Aaleti and Sritharan 2017). The slant shear testing was performed to evaluate the effect of CC compressive strength, interface roughness, curing condition, and pouring sequence on the direct shear transfer behavior. Prismatic specimens consist of normal concrete with five different texture along the interface plane and cast in place UHPC were used for performing the slant shear test. The composite specimen dimensions were 4.5 in. × 6 in. in cross-section and 24 in. long and interface angle of 53.1° with the horizontal axis as shown in Figure 2.22.

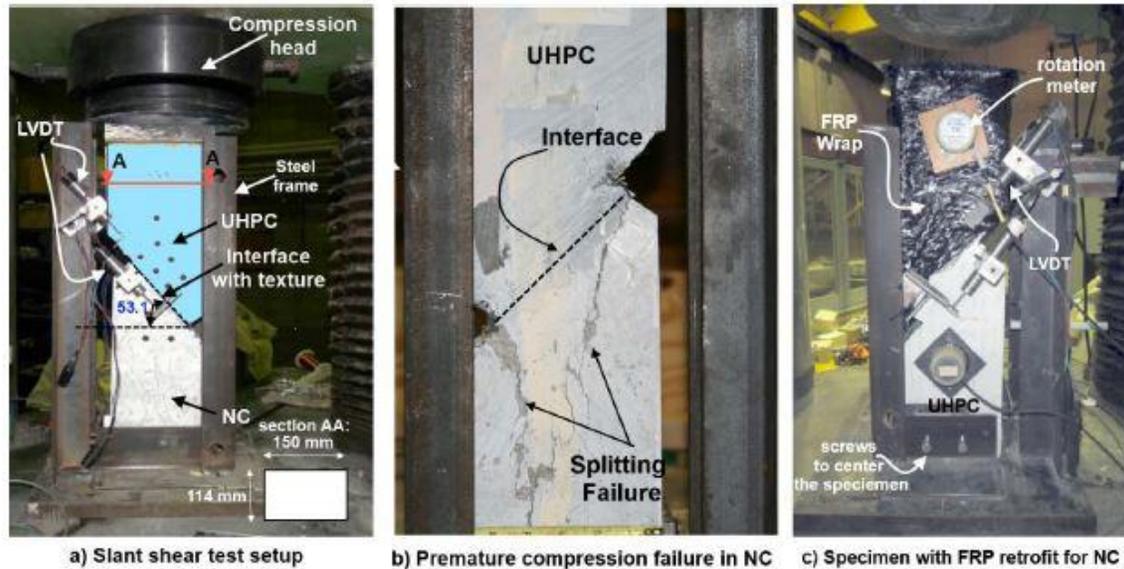


Figure 2.22: Test Setup and Instrumentation of Large Prism Slant Shear Test (Aaleti and Sritharan 2017)

A total of sixty specimens were fabricated with three normal concrete compressive strength, five different textures, and four different curing conditions as shown in Table 2.5. The surface textures were obtained by adding form liners to the interface shear plane. The five surface textures represented low roughness (< 0.06 in.), medium roughness (0.12 in.), and high roughness (0.2 in. to 0.25 in.). The CC sections of composite specimens were cast vertically, and their compressive strength were obtained at 28 days and at the time of slant shear specimen testing. Then, Ductal JS1000 UHPC was used to cast the second section of composite specimens. The texture depth of composite section was measured before pouring the second half to ensure the depth. Based on ASTM C882, a uniaxial compression load was applied at the end of the composite slant shear specimens using a universal testing machine as shown in Figure 2.22. Four linear variable differential transducers (LVDTs) were used to capture the slip at the interface shear plane. Two rotation meters were used to capture any rotation induced by possible eccentricity of loading.

Table 2.5: Summary of UHPC-CC Interface Test Matrix (Aaleti and Sritharan 2017)

Specimen Type	Texture (# of specimens)	Casting Sequence	Target CC Strength
UHPCw-CC5	5 textures (3 per texture)	Wet UHPC over cured CC	5 ksi
UHPCw-CC7	5 textures (3 per texture)	Wet UHPC over cured CC	7 ksi
UHPCw-CC10	5 textures (3 per texture)	Wet UHPC over cured CC	10 ksi
UHPCch-CC5	5 textures (3 per texture)	Wet CC on heat-treated UHPC	5 ksi

Two failure modes were exhibited by the slant shear specimens, bond failure or CC failure, as shown in Figure 2.23. The authors did FRP retrofitting to CC section of some specimens that did not experience significant sliding due to splitting cracks in CC. The interface shear resistance was calculated by dividing the maximum load along the inclined plane by the interface contact area. Figure 2.24 shows

average interface shear resistance of three specimens for different surface texture depths and concrete compressive strength. The interface shear resistance generally increased with the increase of texture roughness and concrete strength. Average interface shear resistance of textures deeper than 0.08 in. satisfied the ACI 546.3R-06 limits. Also, interface shear capacity calculated based on AASHTO (2010) equations were conservative in predicting CC-UHPC interface shear resistance.

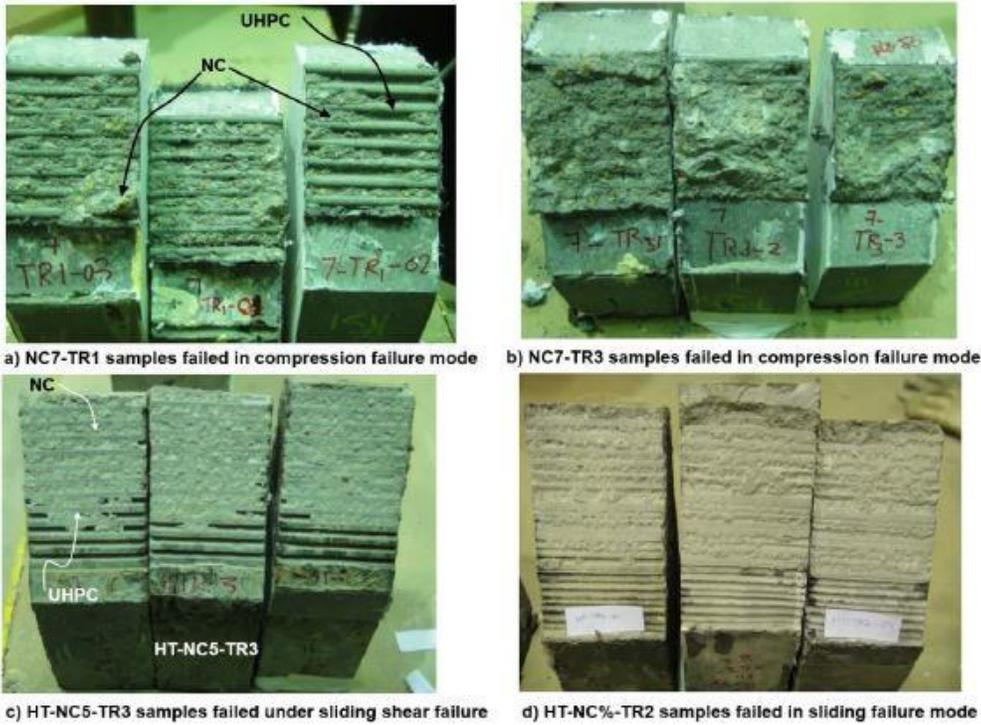


Figure 2.23: Samples of CC-UHPC Interfaces of Specimen with Different Failure Modes (Aaleti and Sritharan 2017)

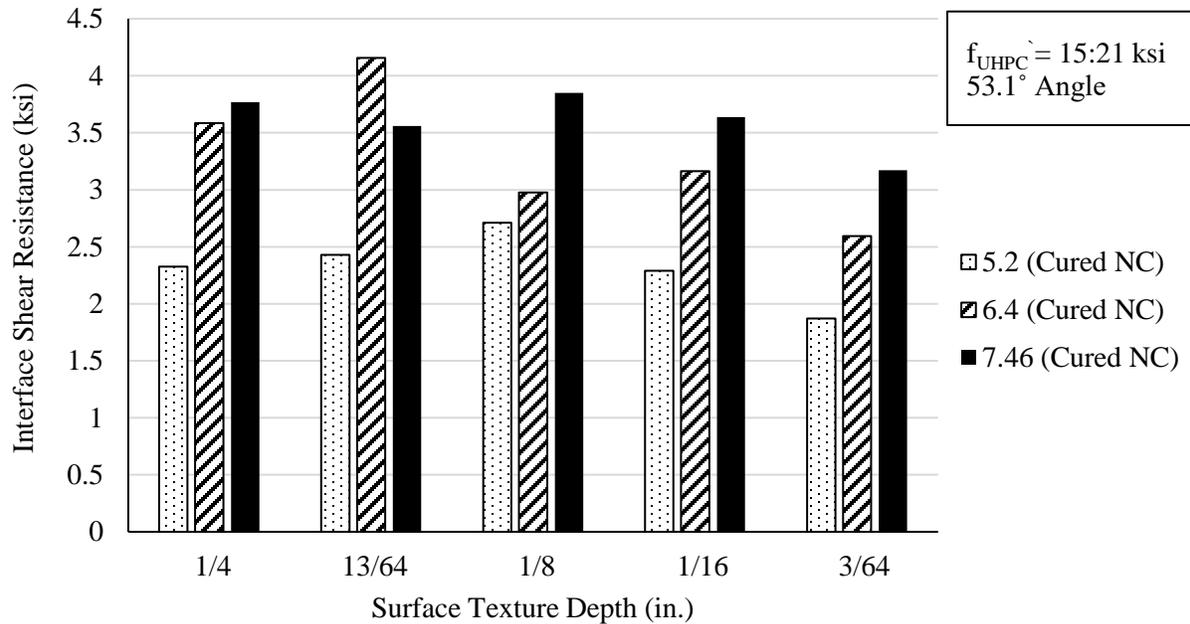


Figure 2.24: Effect of Surface Texture Depth and CC Compressive Strength on Interface Shear Resistance of CC-UHPC (Aaleti and Sritharan 2017)

Jang et al. (2017) conducted vertical shear test on L-shape specimen to evaluate the interface shear resistance of CC-UHPC without interface reinforcement as shown in Figure 2.25. The L-shape specimen dimensions are 5.9 x 11.8 x 25.2 in. with interface shear area of 5.9 x 7.9 in. Five different surface treatments were applied to the CC sections; smooth, water jet, grooved (0.4 in.), grooved (0.8 in.), and grooved (1.2 in.) as shown in Figure 2.25. The UHPC matrix consists of water-to-binder ratio (W/B) of 0.14, type I/II Portland cement, Australian silica sand, and silica with a fiber content of 1.5% of volume. The conventional concrete and UHPC achieved 5.2 and 29.08 ksi respectively. A vertical load was applied on the specimen with a rate of 0.024 in./min. till failure. Four LVDTs were used to capture the horizontal and relative vertical displacement in the L-shape specimen. Figure 2.26 shows the average interface shear resistance of CC-UHPC with different surface treatment. Based on the results, the interface shear resistance of CC-UHPC without interface reinforcement increases with the increase of the surface roughening.

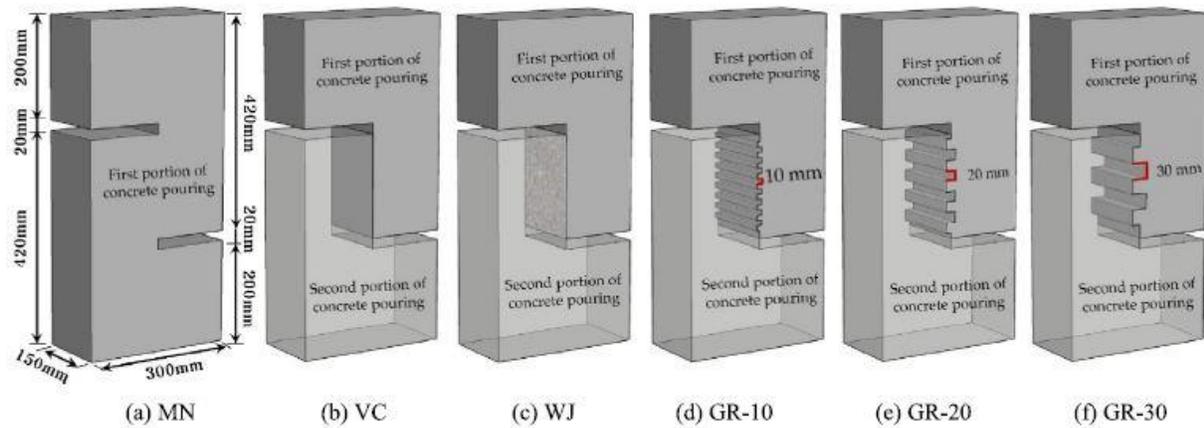


Figure 2.25: L-Shape Specimen Dimensions and Different Surface Treatment of CC-UHPC (Jang et al. 2017)

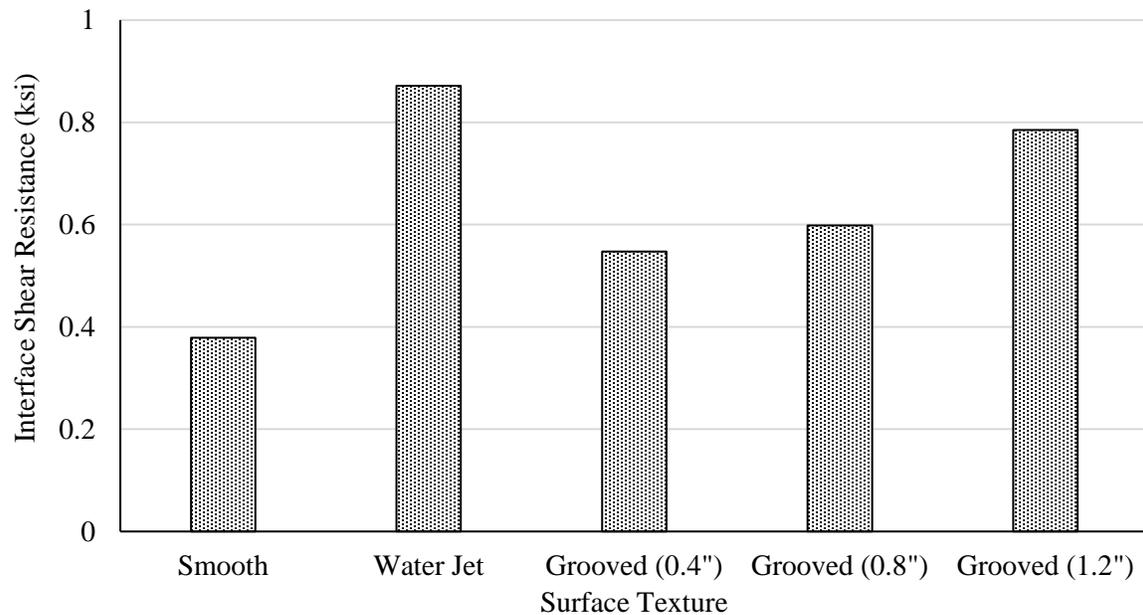


Figure 2.26: L-Shape Test Results of CC-UHPC Specimens (Jang et al. 2017)

2.4.Existing Provisions for Interface Shear Resistance

However, AASHTO LRFD Bridge Design Specifications (2017) does not have equations to predict the interface shear resistance of UHPC, the NF-P-18-710-UHPC (French Code for UHPC) stated using the interface shear resistance equations for CC and stated also two equations for calculating the nominal interface shear resistance of UHPC cast on hardened UHPC based on the interface surface texture as following:

$$V_{Rdi} = cf_{ctk,el}/\gamma_c + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) \leq 1.15\alpha_{cc}f_{ck}^{2/3}/\gamma_c \dots\dots\dots (1)$$

$$V_{Rdi} = cf_{ctk,el}/\gamma_c + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) + (0.35\mu + 0.3) f_{ctfk}/K \cdot \gamma_c \leq 1.15\alpha_{cc}f_{ck}^{2/3}/\gamma_c \dots\dots\dots (2)$$

Where:

c and μ: UHPC cohesion and friction factors depend on interface surface roughening

σ_n: the stress caused by the minimum external axial force across the interface

ρ: interface reinforcement ratio across the interface plane

α: angle of fiber indentation with the interface shear surface of the hardened UHPC as shown in Figure 2.32; 45° < α < 90°.

f_{ctk,el}: characteristic value of the tensile limit of elasticity

f_{ctfk}: characteristic value of the post-cracking strength

f_{ck}: characteristic value of compressive strength

γ_c: partial factor for compressed UHPC (can be reduced to 1.3)

K: fiber orientation factor

The first equation predicts the nominal interface shear resistance for smooth surface and the second one in case of fluted surface. Figure 2.27 shows the limits that must be satisfied to consider the surface is fluted. Also, the depth of indentation “d” must be larger than twice the length of the longest fibers contributing to ensuring non-brittleness. Table 2.6 shows the UHPC cohesion and friction factors with different surface textures.

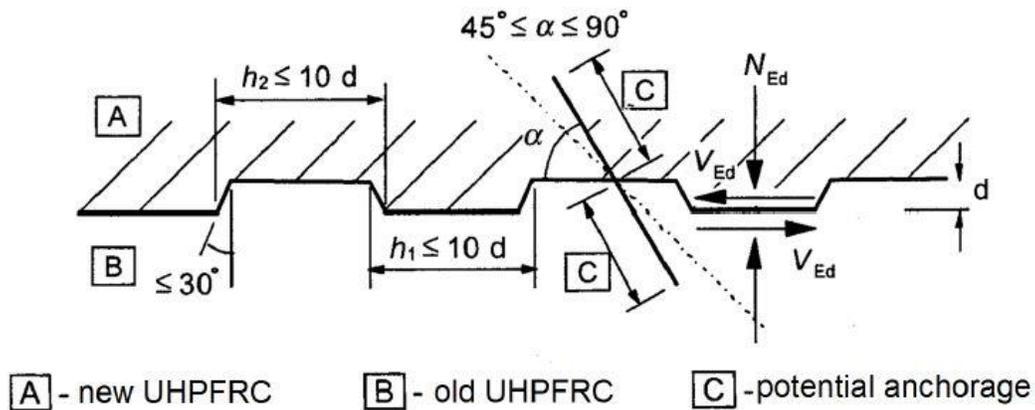


Figure 2.27: Fluted Construction Joint with Indented Fibers (NF-P-18-710-UHPC 2016)

Table 2.6: UHPC Cohesion and Friction Factors of UHPC for Different Surface Textures based on NF-P-18-710-UHPC 2016

UHPC Surface Texture	UHPC Cohesion Factor (c)	UHPC Friction Factor (μ)
Formed Clean Surface	0.025-0.10	0.5
Un-Formed Clean Surface	0.2	0.6
Roughened Clean Surface with Form liner	0.4	0.7
Clean Fluted Surface	0.5	1.4

Chapter 3. Proposed Deck-To-Girder Connection

3.1. Introduction

This chapter presents a new UHPC connection between precast concrete deck panels and NU precast/prestressed concrete girders that eliminates any changes to girder design/production and any possible conflict between deck and girder reinforcement. The initial design was forming trough through the width of the panel which consume huge quantities of UHPC. This design was updated to using a corrugated shear pockets with No. 5 loops as final design. A design example and the experimental investigation procedure are presented.

3.2. Initial Design

Based on the literature review, the current precast concrete deck-to-girder connections using UHPC consist of open longitudinal joints or covered longitudinal troughs with exposed aggregate finish and grouting holes every 24 in. over each girder line. These two systems consume large quantities of UHPC to fill the joints and haunches that impact significantly the system economics based on the high price of UHPC. Also, using opened/covered longitudinal joints prevent transverse prestressing of concrete deck panels that limits the size of precast panels.

The initial design was forming trough along the width of the panel and casting UHPC through grouting holes to fill the trough and haunch (Abo El-Khier, et al. 2018). Figure 3.1 and Figure 3.2 show two alternatives for forming the haunch: option I requires continuous deck support system and large quantity of UHPC, and option II requires discrete deck support system, compressible material, and smaller quantity of UHPC. Figure 3.3 shows a preliminary design of the panel trough proposed for this application. Figure 3.4 shows two alternatives for panel reinforcement and pre-tensioning strands: Option I with solid concrete zones at the panel ends and middle to provide two layers of pre-tensioning strands at these locations; and Option II with three equal troughs and two layer of pre-tensioning strands at the solid concrete zones. The main advantage of these options over the continuous trough concept presented in the literature is the use of pre-tensioning strands to transversely prestress the precast concrete panels, which minimizes panel cracking during handling and transportation.

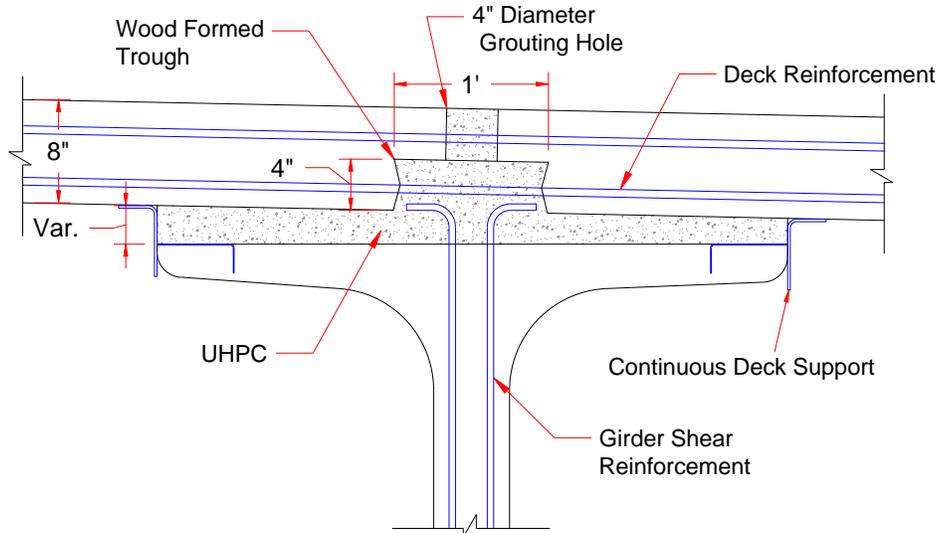


Figure 3.01: Initial Design Connection (Option I)

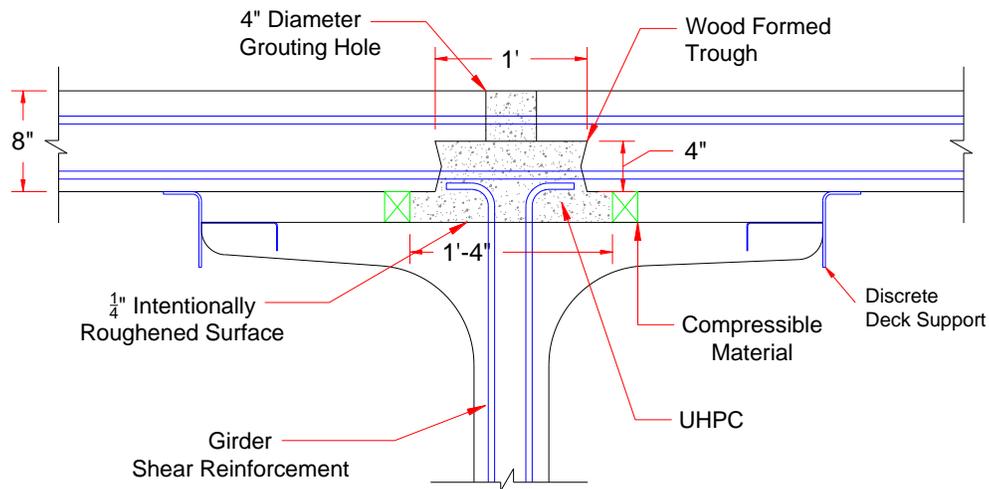


Figure 3.02: Initial Design Connection (Option II)

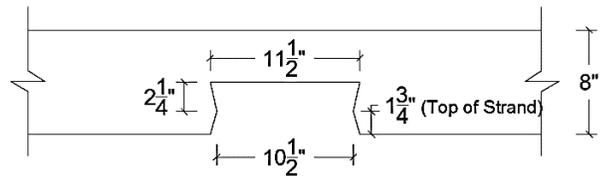


Figure 3.03: Initial Design Proposed Panel Trough.

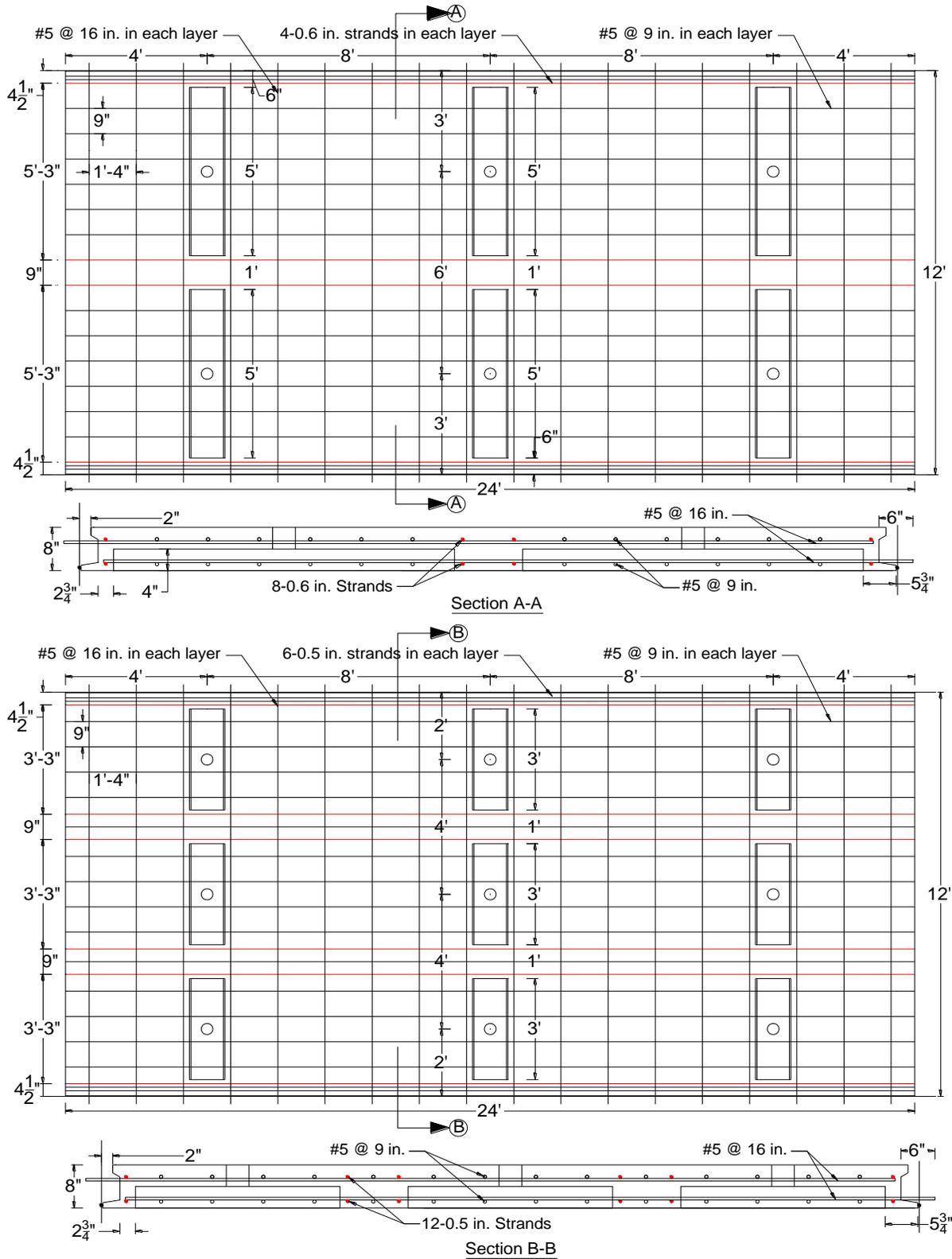


Figure 3.4: Alternatives for Panel Reinforcement and Pre-Tensioning.

3.3. Proposed Deck-to-Girder Connection Using UHPC

The initial design required a huge amount of UHPC to fill the trough and haunch that leads to making the proposed system not economic. Also, the deck panel solid parts would exhibit high stress concentration due to the pre-tensioned strands. To simplify the initial design, the haunch is eliminated and replaced with discrete round shear pockets. Figure 3.5 shows the proposed precast concrete deck-to-girder connection using UHPC supported by precast/prestressed concrete girders. In this connection, discrete round shear pockets 4 – 8 in. in diameter are formed in the deck panels every 2 - 4 ft. over each girder line. The diameter and spacing of these pockets are determined based on the interface shear demand. Girder interface shear reinforcement is the same as it is in case of CIP concrete bridge decks but lowered to remain below the soffit of deck panels. Once all panels are installed at the desired elevation using support angles or leveling bolts, a loop bar is inserted in each shear pocket to cross the interface between the two components. The shear pockets and haunches are, then, filled with UHPC cast from the shear pocket openings to connect the two components and achieve the composite section. The side surface of the shear pockets should be roughened using either form liner or exposed aggregate to provide adequate bond between UHPC and the deck panel concrete. Also, blocks of compressible material are recommended as shown in Fig. 3.5 to form the haunch area and reduce the quantity of field cast UHPC. The same concept can be used to connect precast concrete deck panels to steel girders with conventional shear studs as shown in Figure 3.6, which is not the focus of this study .

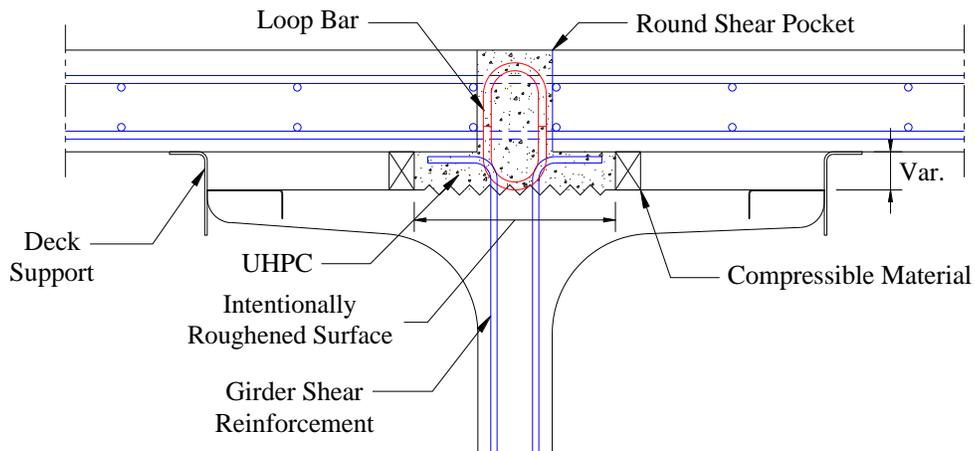


Figure 3.5: Proposed Precast Concrete Deck-To-Concrete Girder Connection

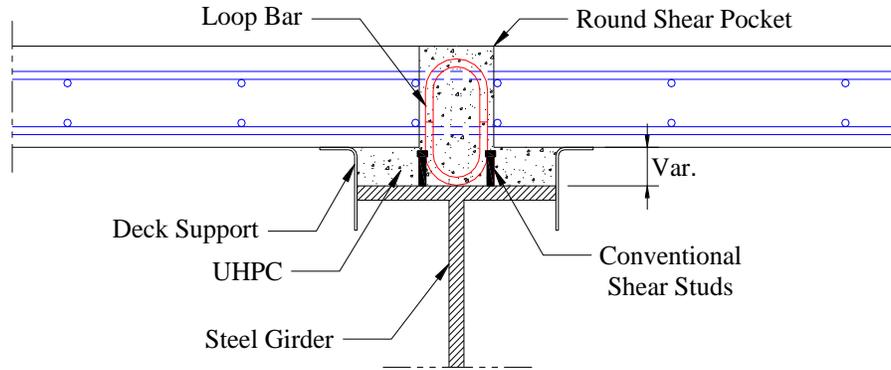


Figure 3.6: Proposed Precast Concrete Deck-To-Steel Girder Connection

The proposed connection allows transverse prestressing of precast deck panels as shown in Figure 3.7. Prestressing the deck panels decreases significantly the deck reinforcement besides eliminating the need for forming the overhangs. Also, prestressing deck panels allow using large panels that reduce the construction time and minimize the number of open joints that need to be ground for leveling the surface. The round shape of proposed shear pocket eliminates any tolerance limits for adding the loop bar and keep it in position as shown in Figure 3.7.

Two critical interface shear planes exist in this connection as shown in Figure 3.8. The first plane is at the girder top surface between fresh UHPC and hardened conventional concrete (CC-UHPC), which is intentionally roughened surface as a common practice. The second plane is at the soffit of the deck panels across the monolithic UHPC. The loop bar placed in each pocket crosses the second plane to enhance its interface shear resistance. Also, the roughened side surface of the shear pocket prevents pocket pull-out from the deck panel concrete. Since current AASHTO LRFD Bridge Design Specifications 2017 does not provide equations for predicting the interface shear resistance of either monolithic UHPC or CC-UHPC, experimental investigations are conducted to understand the new connection behavior and predict the interface shear resistance of connection.

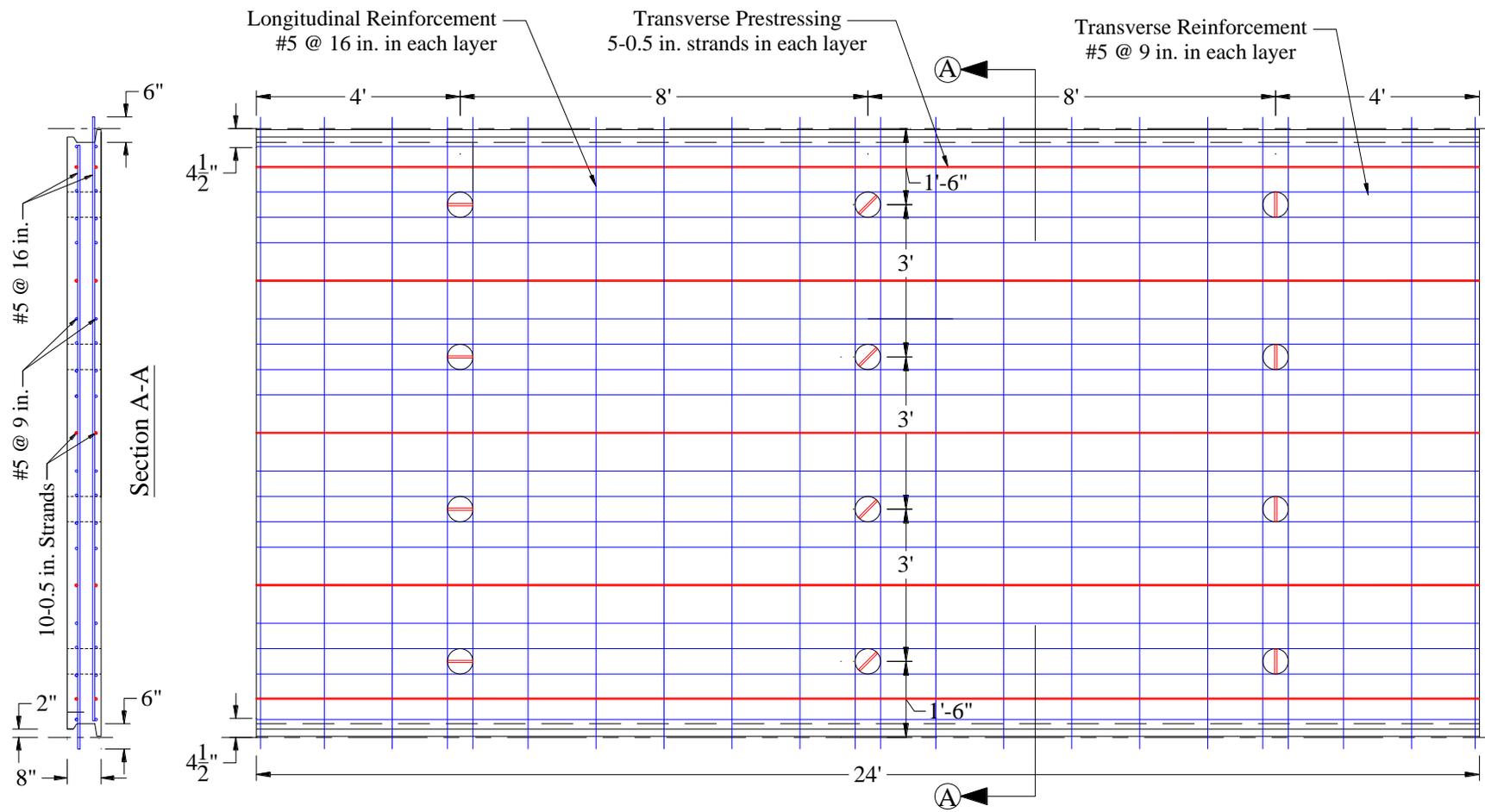


Figure 3.7: Panel Reinforcement and Pre-Tensioning for Proposed Connection.

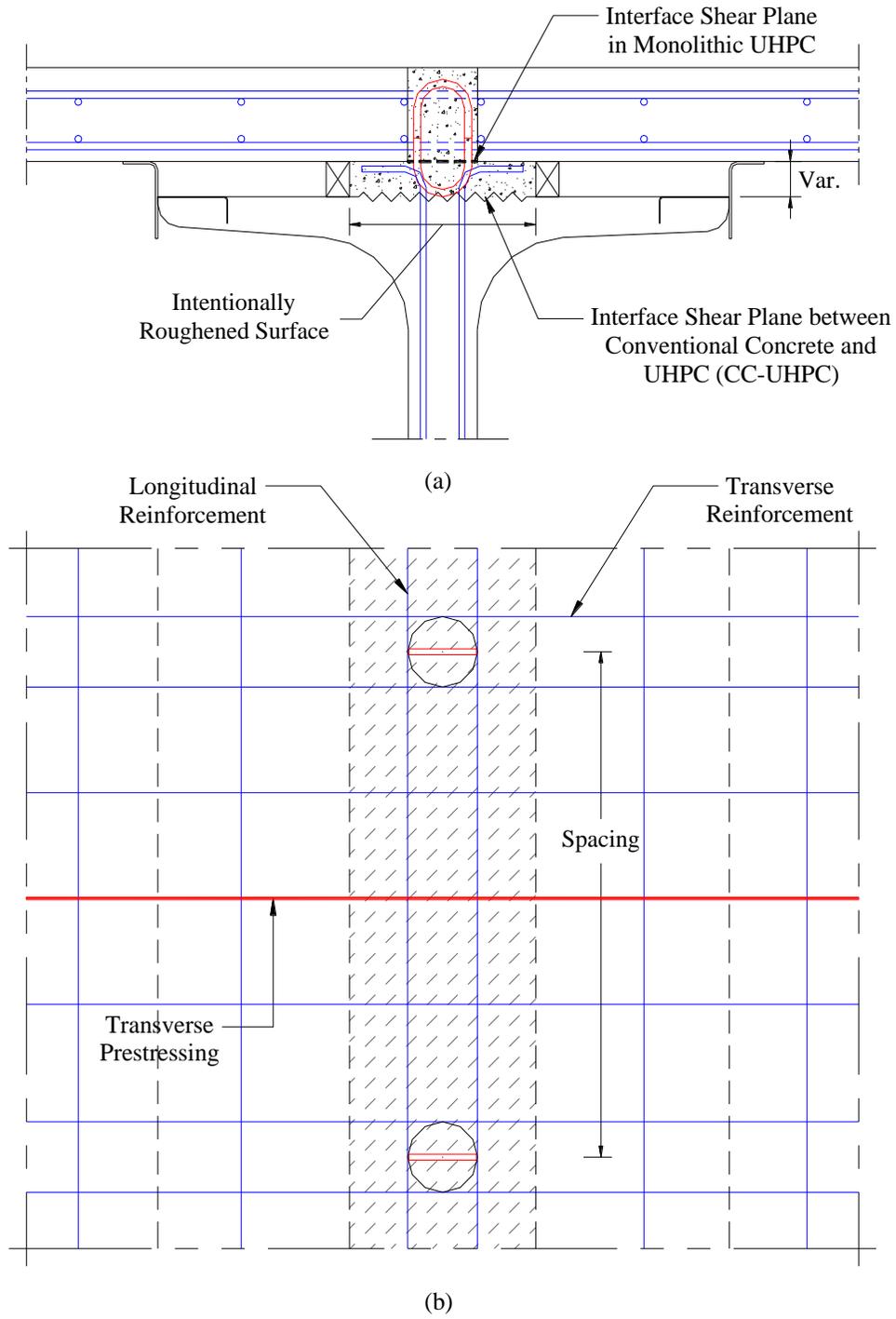


Figure 3.8: Interface Shear Resisting Area; (a) at the Top of the Concrete Girder and (b) at the Soffit of the Deck Panels

3.4. Construction Sequence of New Connection

The construction sequence of the new precast concrete deck-to-concrete girder using UHPC is presented in the following steps as shown in Figure 3.9:

1. Fabricate precast/prestressed concrete girders with conventional shear reinforcement and roughened the girder top with the design width along the girder line.
2. Fabricate precast/prestressed concrete deck panels with discrete round shear pockets at designed spacing, roughened side surface of the shear pockets using either form liner or exposed aggregate and side rails, which also host the lifting inserts.
3. Erect all girders and check their alignment using a digital template to ensure the girder spacing within the specified tolerance.
4. Form, reinforce, and pour end and intermediate diaphragms up to the girder top flange.
5. Conduct shim shots on the edges and center of each girder line to determine the actual profile of the cambered girders prior to panel erection.
6. Place blocks of compressible material to form the haunch area, achieve the desired deck elevation and a minimum haunch thickness of 4 in.
7. Attach extruded polystyrene panels to the diaphragm concrete between girders to fill the gap between the deck panels and diaphragm concrete between girders.
8. Place negative reinforcement bars extending from top flange of each girder over the intermediate support. Avoid conflict between reinforcement bars and strands using supporting chairs to bars.
9. Place each deck panel on compressible material blocks using lifting points.
10. Fill the gaps between adjacent deck panels using backer rod and clean/moist the joint surface prior to cast UHPC.
11. Form ½” plywood sheets around the transverse joints and use blocks/buckets to cover the filled pockets to prevent UHPC from overflow.
12. Pour UHPC to fill haunch, round shear pockets, and transverse joints between deck panels and rails. Pouring should continue until the UHPC overflow from every pocket.
13. Grind the top surface of UHPC before achieving 10 ksi compressive strength to level deck top surface.
14. Form, reinforce, and pour approach slabs

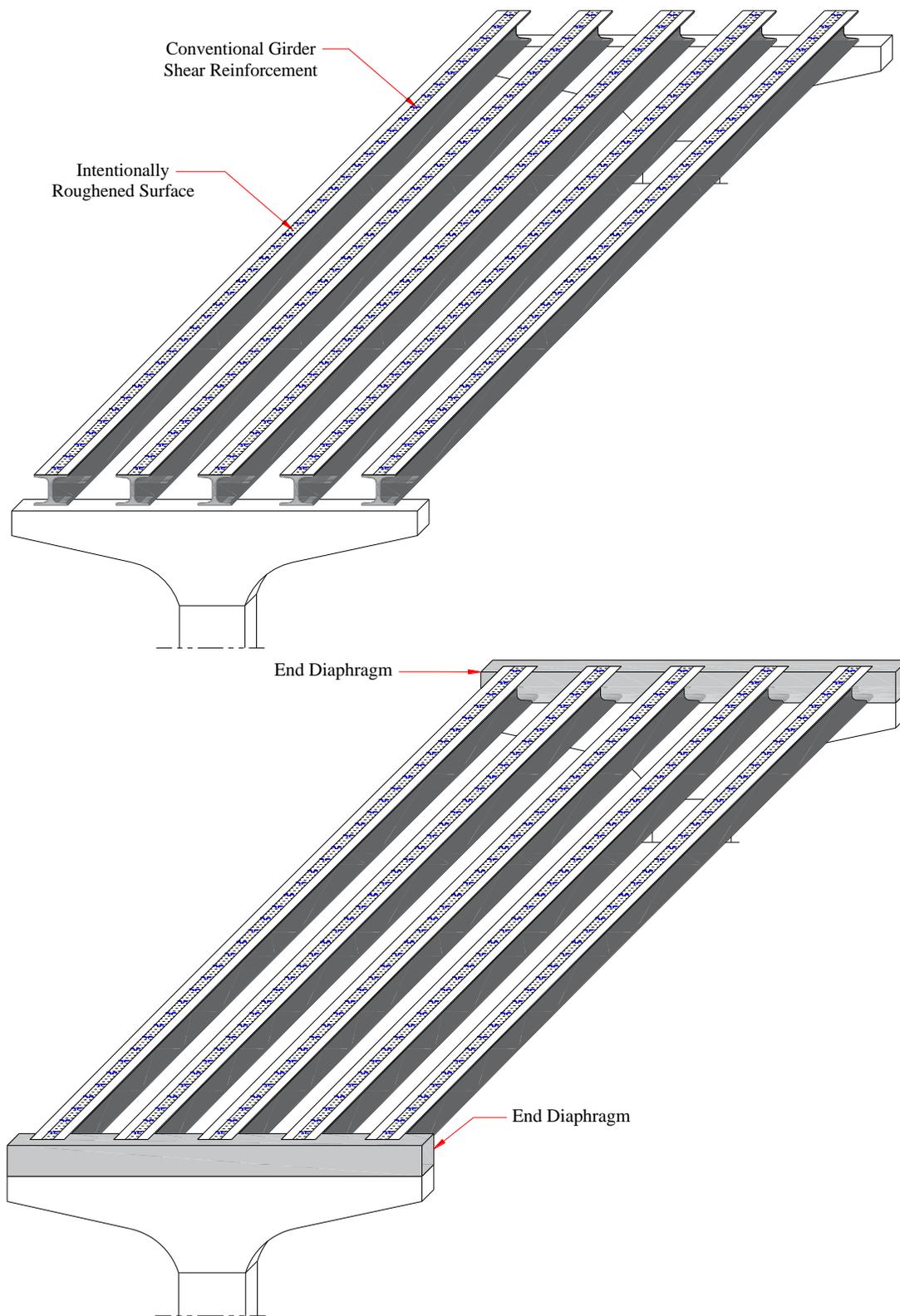


Figure 3.9.1: Construction Sequence of the Proposed Precast Concrete Deck-to-Concrete Girder Connection Using UHPC

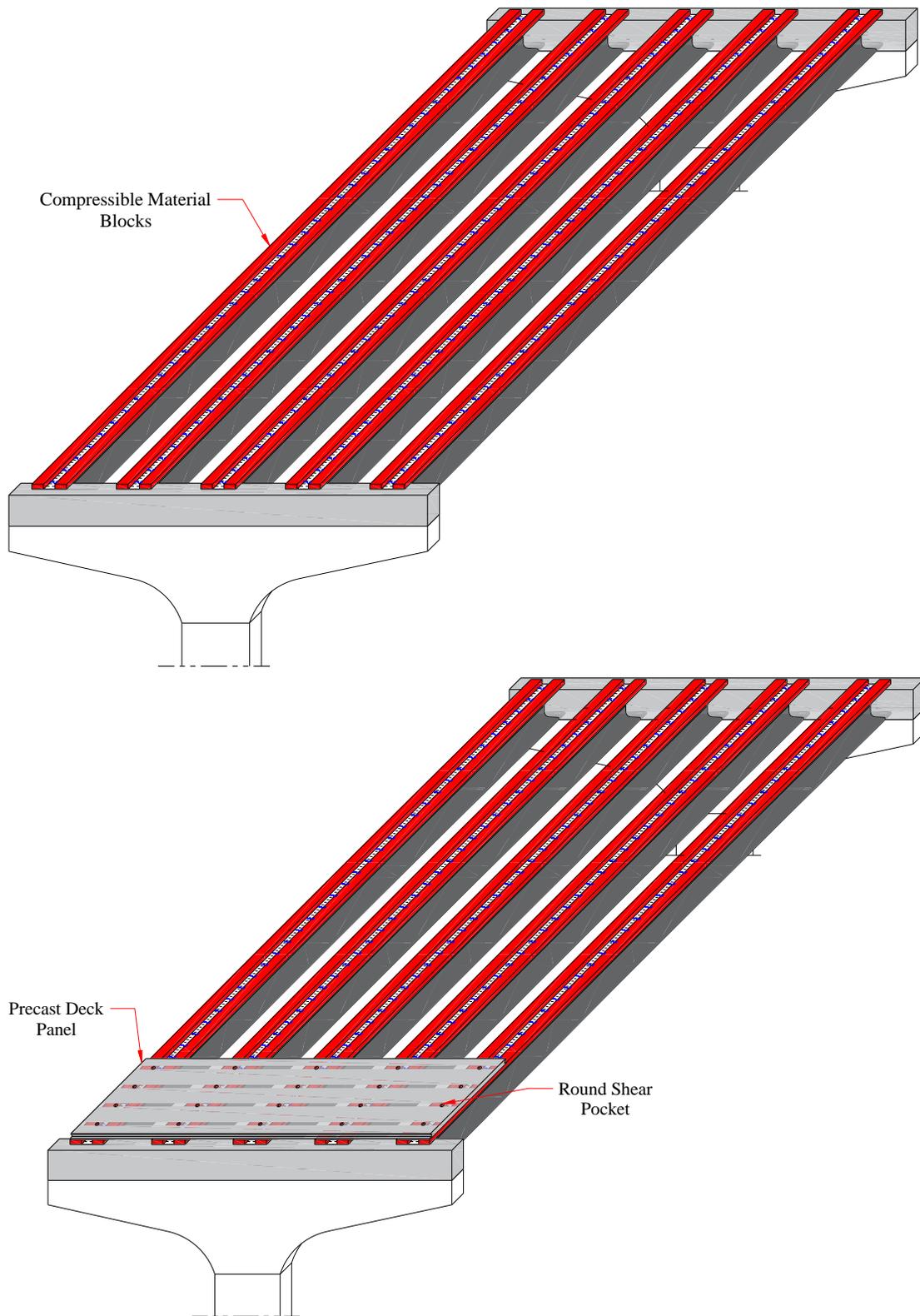


Figure 3.9.2: Construction Sequence of the Proposed Precast Concrete Deck-to-Concrete Girder Connection Using UHPC

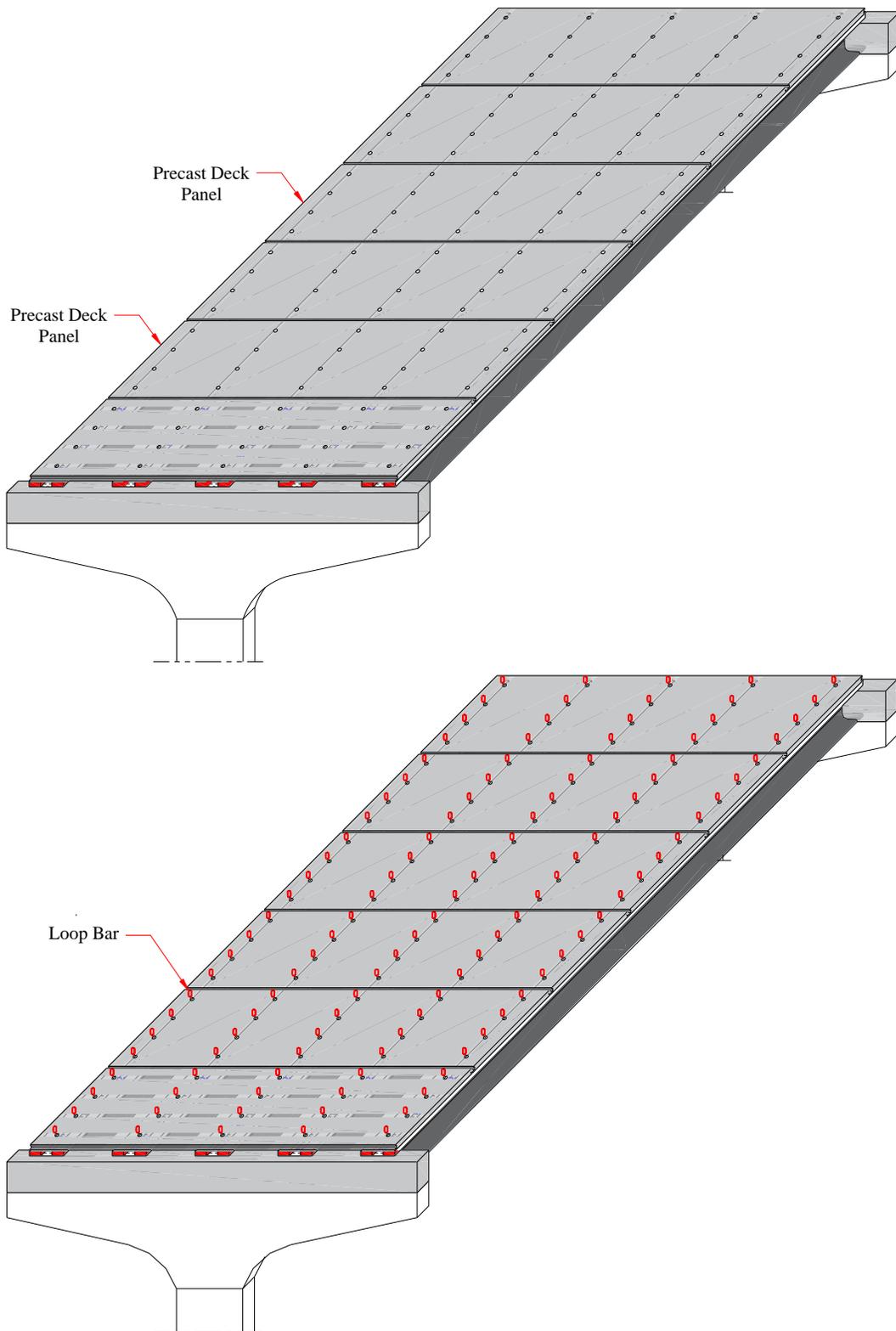


Figure 3.9.3: Construction Sequence of the Proposed Precast Concrete Deck-to-Concrete Girder Connection Using UHPC

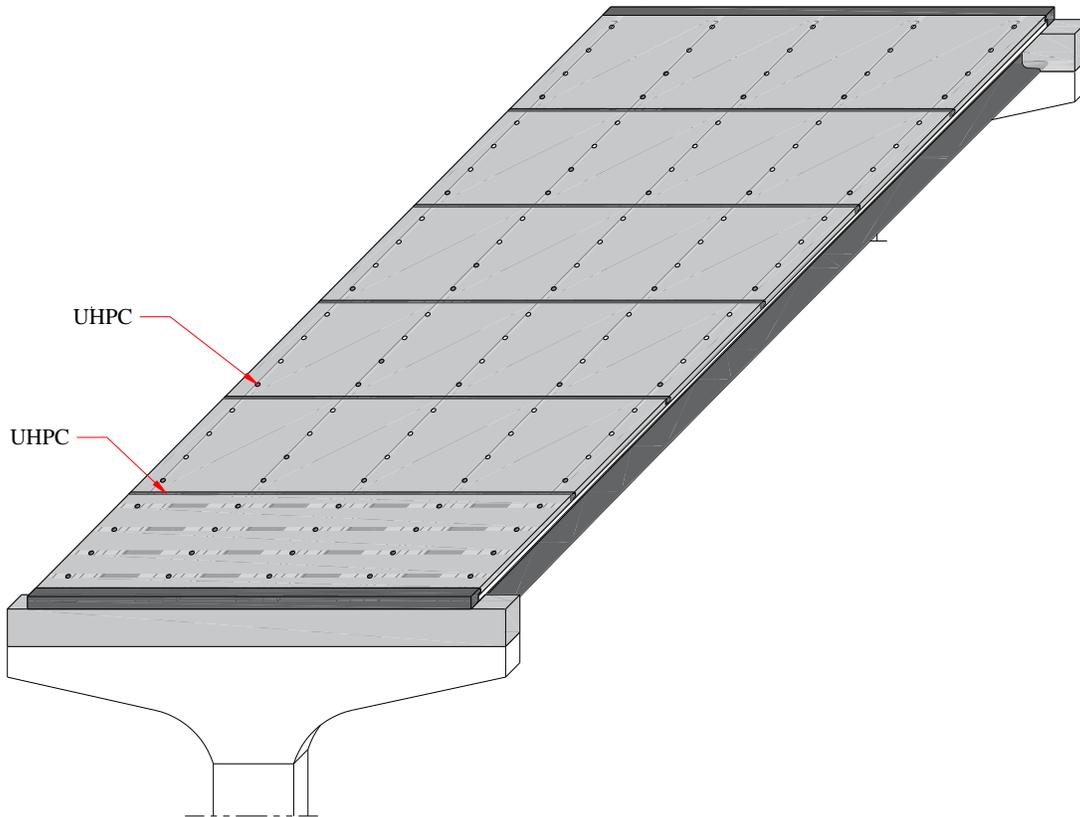


Figure 3.9.4: Construction Sequence of the Proposed Precast Concrete Deck-to-Concrete Girder Connection Using UHPC

3.5. Study Methodology

The study methodology includes two stages; experimental investigation, and design procedure. Two different UHPC mixes are used in evaluating the performance and constructability of proposed connection; commercial and non-proprietary UHPC mixes with 2% steel fiber content by volume. The experimental investigation consists of small-scale testing and full-scale push-off test. Small-scale testing is conducted to evaluate the interface shear resistance at the connection critical sections; direct shear and L-shape push-off tests for monolithic UHPC, and slant shear and L-shape push-off tests for CC-UHPC. Double shear push-off and full-scale push-off specimens simulating an actual connection are designed and tested to obtain the interface shear resistance of the new connection and evaluate connection constructability. Finally, an example bridge is presented to demonstrate connection design using the results of the experimental investigation. The following flowchart shows the study methodology sequence:

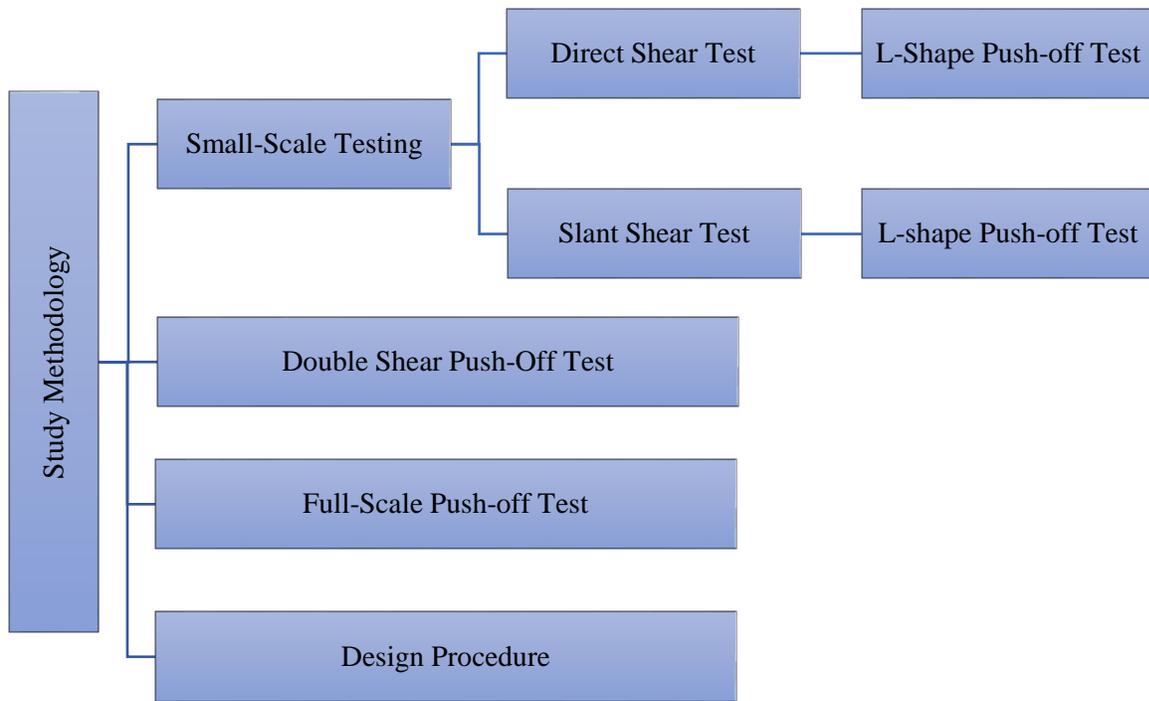


Figure 3.10: Study Methodology for Evaluating Proposed Connection.

Chapter 4. Experimental Investigation

4.1. Introduction

This chapter illustrates the experimental investigation procedure, small-scale and full-scale testing, to evaluate the interface shear resistance of monolithic UHPC and of fresh UHPC cast on hardened conventional concrete (CC-UHPC). Direct shear, L-shape push-off, double shear push-off tests were conducted to evaluate interface shear resistance of monolithic UHPC. The literature review conducted on interface shear resistance of CC-UHPC was summarized and analyzed to propose prediction equations. Then, slant shear test and L-shape push-off test were conducted to evaluate and validate these equations. The constructability and structural performance of the proposed connection was investigated through full-scale push-off tests. The full-scale specimens were designed to use 3 ft. spacing between shear pockets.

4.2. Material Properties

The experimental program was conducted using a non-proprietary UHPC mix as a primary mix and commercially available UHPC mix for some specimens. Table 1 shows the non-proprietary UHPC (UNL UHPC) mix proportions that was designed by University of Nebraska-Lincoln (UNL) research team and sponsored by Nebraska Department of Transportation (NDOT). Straight stainless-steel micro-fibers, 0.50 in. length and 0.078 in. diameter, were used in the UHPC mixes with a dosage of 2% by volume. UNL UHPC provides 18 ksi compressive strength, 6377 ksi modulus of elasticity, 4.88 ksi peak flexural strength at 28 days. The UNL UHPC mixing design and mechanical properties can be found in (Mendonca, F., Abo El-Khier, M., Morcou, M., and Hu, j., Feasibility study of development of UHPC for highway bridge applications in Nebraska, Report No. M072, unpublished report). ASTM A615 Grade 60 uncoated rebar was used for specimens and interface reinforcement.

Table 4.1: UNL UHPC Mix Proportions

UHPC mix	Constituent, Ib/yd ³						
	Cement Type I/II	Silica Fume	Slag	#10 Sand	Water/Ice	HRWRA ^a	Steel Fibers
UNL UHPC	1178	152.7	570	1663.7	317	61	263

^aHigh Range Water Reducer Admixture

4.3. Evaluate Interface Shear Resistance of Monolithic UHPC

The first interface plane is located at the soffit of deck panel in the UHPC shear pocket round cross-section. The interface shear resistance of monolithic UHPC was evaluated through direct shear, L-shape push-off, and double shear push-off tests with and without interface reinforcement as following:

4.3.1. Direct Shear Test

A direct shear test was conducted to evaluate the interface shear resistance of monolithic UHPC without interface reinforcement using 2x2x6 in. prismatic specimens. The specimens were cast from one end in long forms to allow UHPC to flow and align the fiber along the form. The specimens were stripped out of the forms after one day and submerged in lime-saturated water till the day of testing. Then, the specimens were cut using wet saw to the desired length. A steel loading frame was used to apply double shear loading to the specimens as shown in Figure 4.1. A displacement-controlled loading rate of 0.05 in./min. was applied till failure.

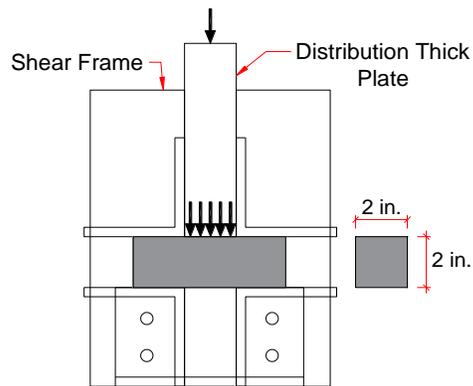


Figure 4.01: Direct Shear Test Setup.

A total of 26 specimens were tested at different compressive strengths of UHPC. All the specimens exhibited double shear failure as shown in Figure 4.2. The obtained direct shear strengths were calculated by dividing the applied load by the double shear areas. The average direct shear strength, of minimum three specimens, ranged from 4.0 to 5.95 ksi as the average compressive strength of UHPC ranged from 11.8 to 23.4 ksi. Figure 4.3 shows the obtained direct shear test results and their comparison to the literature.



Figure 4.02: Double Shear Failure Mode of Direct Shear Test Specimen.

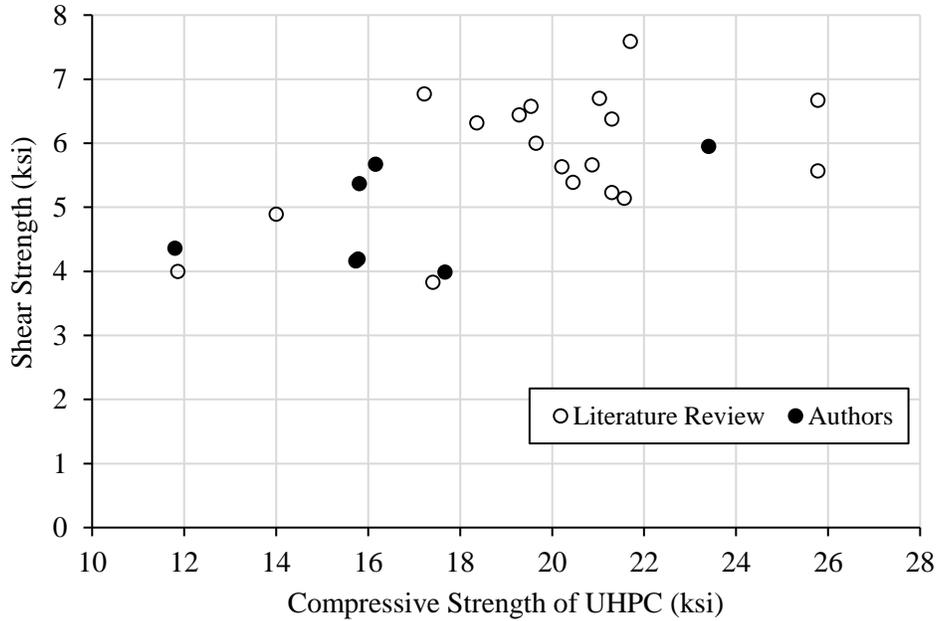


Figure 4.03: The Obtained Direct Shear Test Results and Their Comparison to the Literature.

Figure 4.4 shows the average direct shear strengths of UHPC with different flowability: low (≤ 8 in. spread), medium (8-10 in. spread), and high (>10 in. spread). The flow test was conducted using a standard flow table with diameter of 10 in. according to ASTM C230; specified by ASTM C1856. Specimens with low and medium flowability had almost similar direct shear strength, while specimens with high flowability had 30% less direct shear strength that could be attributed to fiber segregation.

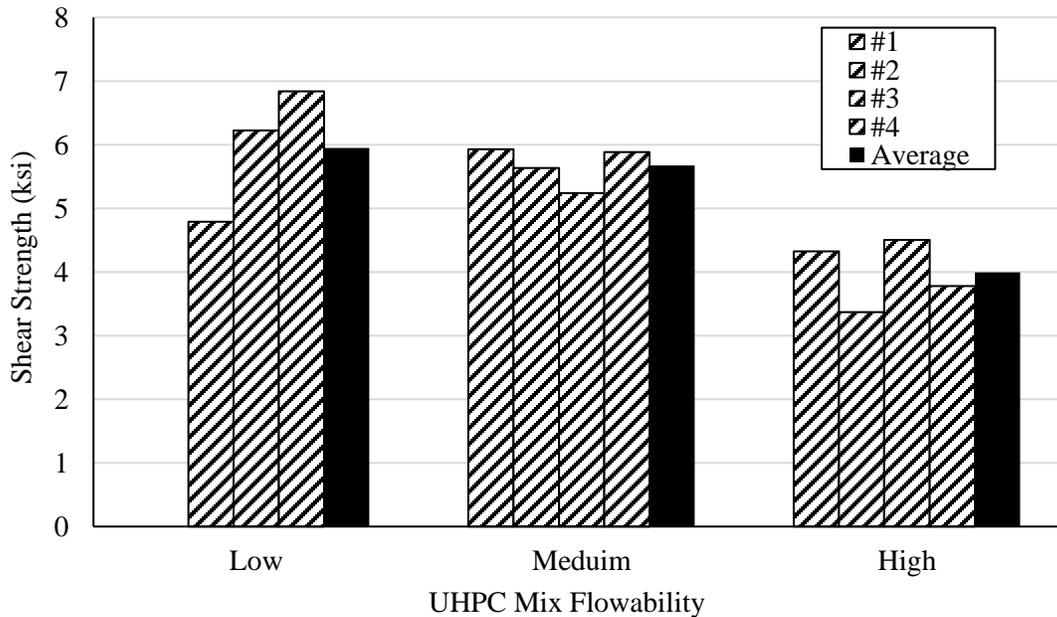


Figure 4.04: Effect of Flowability on Direct Shear Test Results.

It should be noted that direct shear test data shows much higher shear strength than those obtained from L-shape test literature. This difference could be attributed to the small size of the specimens and the corresponding high variability in fiber distribution. Also, the nature of the direct shear test results in different load paths as a compression struts from the applied load and the supports, which does not represent the true shear strength of UHPC.

4.3.2. L-Shape Push-off Test

L-shape push-off test was conducted to investigate the interface shear resistance of monolithic UHPC. The L-shape specimens were casted horizontally, stripped out of forms after one day, and covered with plastic till the testing day. Figure 4.6 and Figure 4.7 show the L-shape specimen details and test setup. The relative vertical displacement (slippage) and relative horizontal displacement between two L-sections at interface plane were captured using four LVDTs; two LVDTs for each side as shown in Figure 4.7. A shear load rate of 600 Ib/sec. was applied till failure using a hydraulic ram after being aligned with the interface plane. The applied load was measured using a pressure transducer attached to the ram. The specimens were label using the form UHPC-MON-A%#B where MON is cast monolithically, A is the interface reinforcement ratio, and #B is the specimen number.



Figure 4.05: L-Shape Push-off Specimen Preparation.

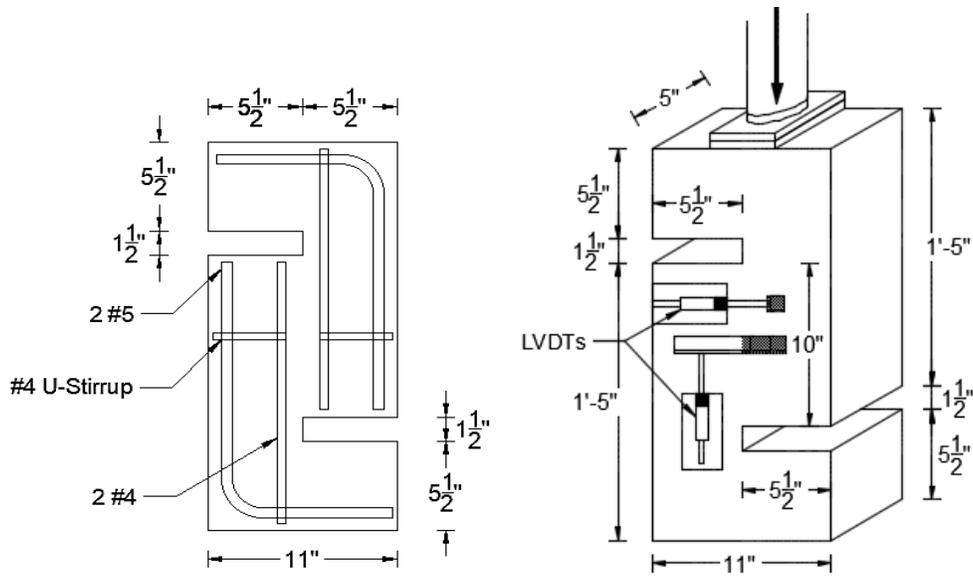


Figure 4.06: L-Shape Push-off Specimen Details

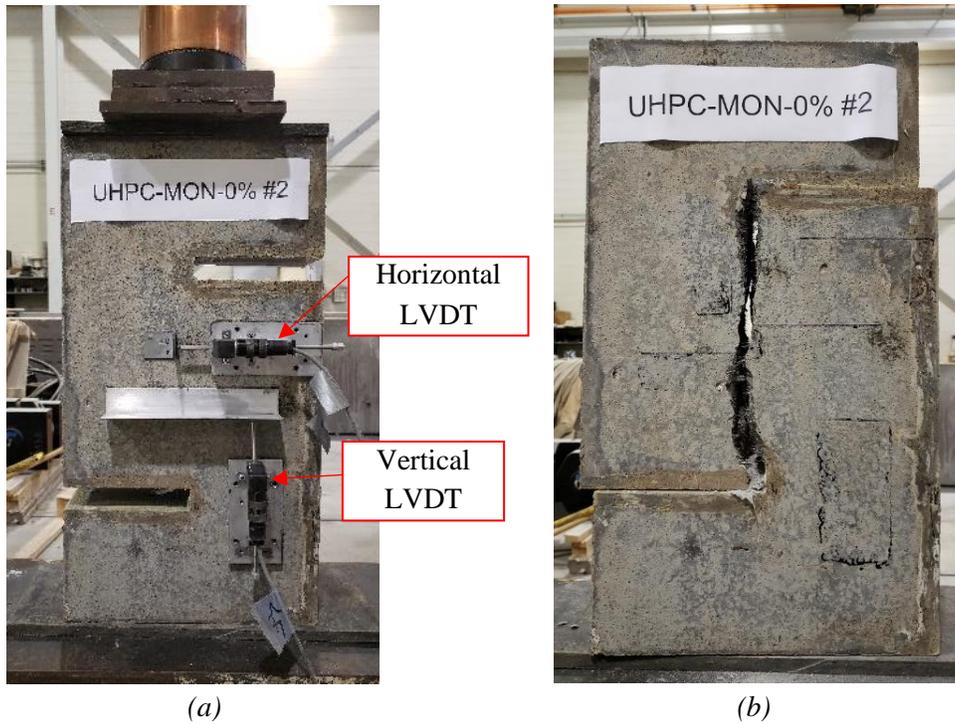


Figure 4.07: L-Shape Push-off Test; (a) Test Setup, and (b) Failure Mode.

All the specimens exhibited interface shear failure as shown in Figure 4.7. The interface shear resistance was calculated by dividing the applied load by interface shear area (50 in.²). The specimens #1 to #3 had a compressive strength of 25.3 ksi and specimens #4 and #5 had a compressive strength of 17.1 ksi. The average interface shear resistance of monolithic UHPC are 2.42 ksi and 2.0 ksi at UHPC compressive strength of 25.3 ksi and 17.1 ksi, respectively. The slippage did not exceed 0.01 in. at the peak

load as shown in Figure 4.8(a). Figure 4.8(b) shows the effect of fibers crossing the interface that act like stitches and provides ductile behavior at the peak load without interface without reinforcement. This ductile behavior mainly controlled by the fiber content in UHPC mix which might change with different fiber content.

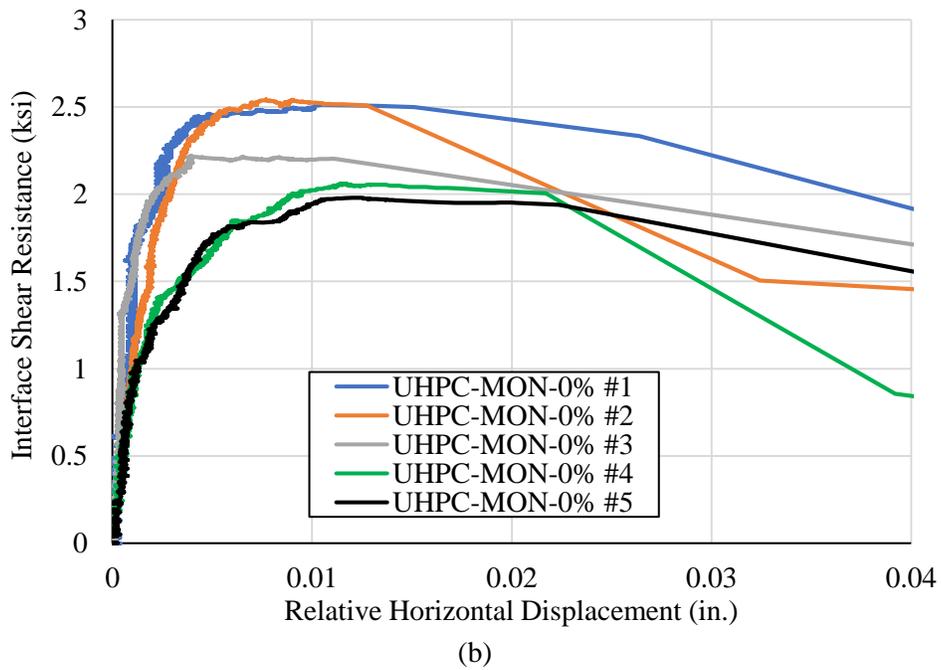
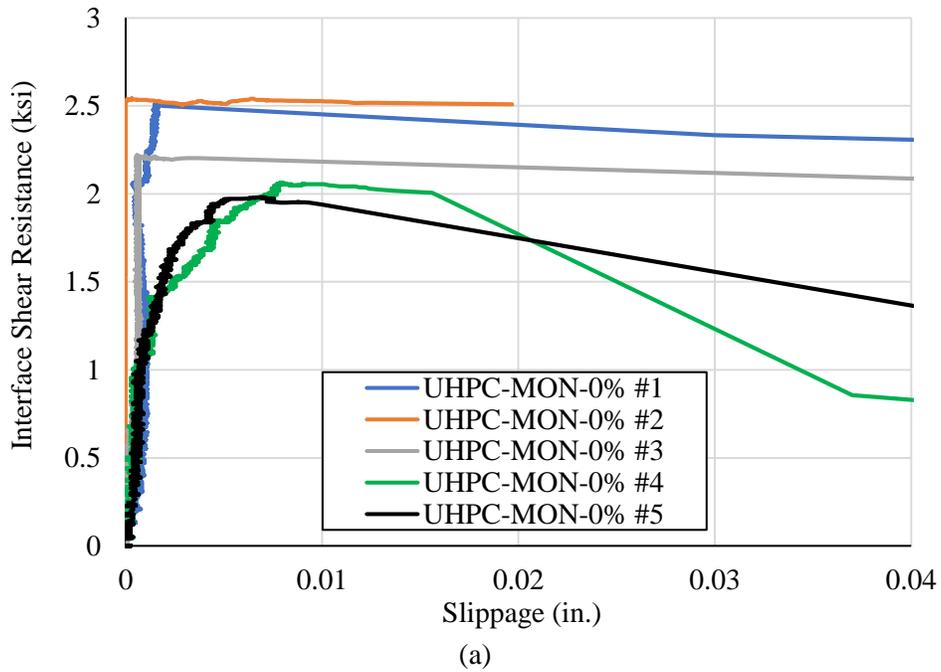


Figure 4.08: Relative Displacement between Monolithic L-Shape Push-off Sections; (a) Slippage, and (b) Relative Horizontal Displacement.

Figure 4.9 shows the results of L-shape push-off tests and their comparison with similar testing in the literature. The L-shape test results had better consistency and less scatter than those of direct shear tests. It also shows that the interface shear resistance of monolithic UHPC depends on the compressive strength of UHPC as the trendline gives good correlation with the square root of UHPC compressive strength. The interface shear resistance of monolithic UHPC can be predicted using the proposed cohesion factor as follows, which is much higher than that of monolithic CC (0.4 ksi):

$$c = 0.49 \sqrt{f'_{UHPC}} \quad (\text{ksi})$$

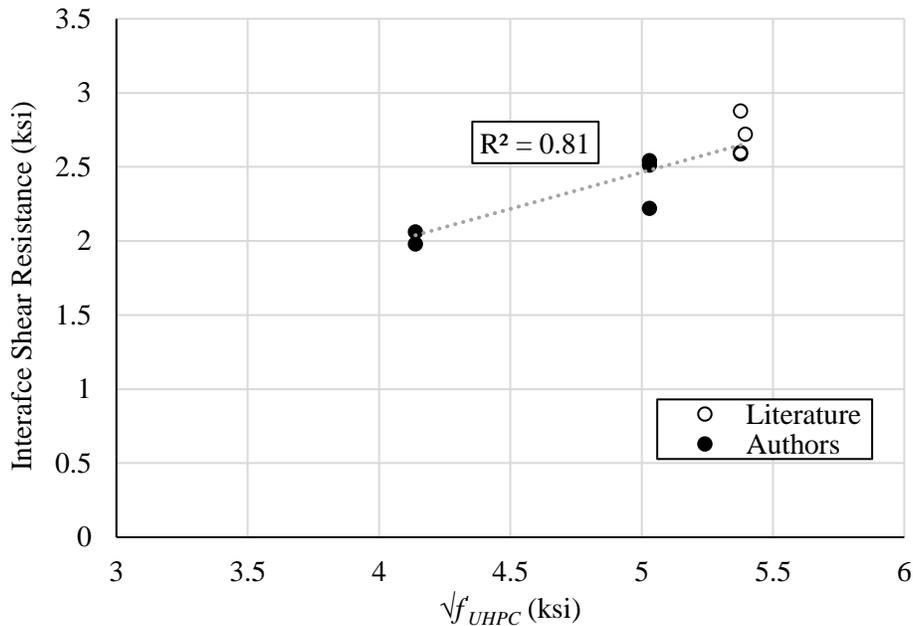


Figure 4.09: L-Shape Push-off Test Results of Monolithic UHPC and their Comparison to the Literature.

4.3.3. Double Shear Push-Off Test

The interface shear resistance of monolithic UHPC with interface reinforcement was evaluated through performing a large-scale double shear push-off test. The double shear push-off specimens were designed to mimic the proposed connection with 6 in. diameter round shear pocket and embedded No. 5 loop bar. Two 20x20x8 in. conventional concrete (CC) slabs, with 6 in. diameter shear pocket in the center, were cast to be used for applying the shear load. Two reinforcement mats of 8#4 for each were used to enhance the capacity of concrete slab. The shear pocket was formed using a corrugated plastic pipe, provides 0.25 in. roughened side surface, to prevent the UHPC section from lateral movement. Two 16x20x4 in. UHPC slabs were cast on the sides of precast CC slab. Each UHPC slab had two U-shape #4 bars to enhance the slab capacity. A #5 loop bar was added to shear pocket through the UHPC slabs before casting. The UHPC slabs were 2 in. below the top of CC slab to allow vertical movement. Figure 4.10

shows the double shear push-off specimen details and reinforcement. The specimens were label using the form UHPC-MON-A%#B where MON is cast monolithically, A is the interface reinforcement ratio, and #B is the specimen number.

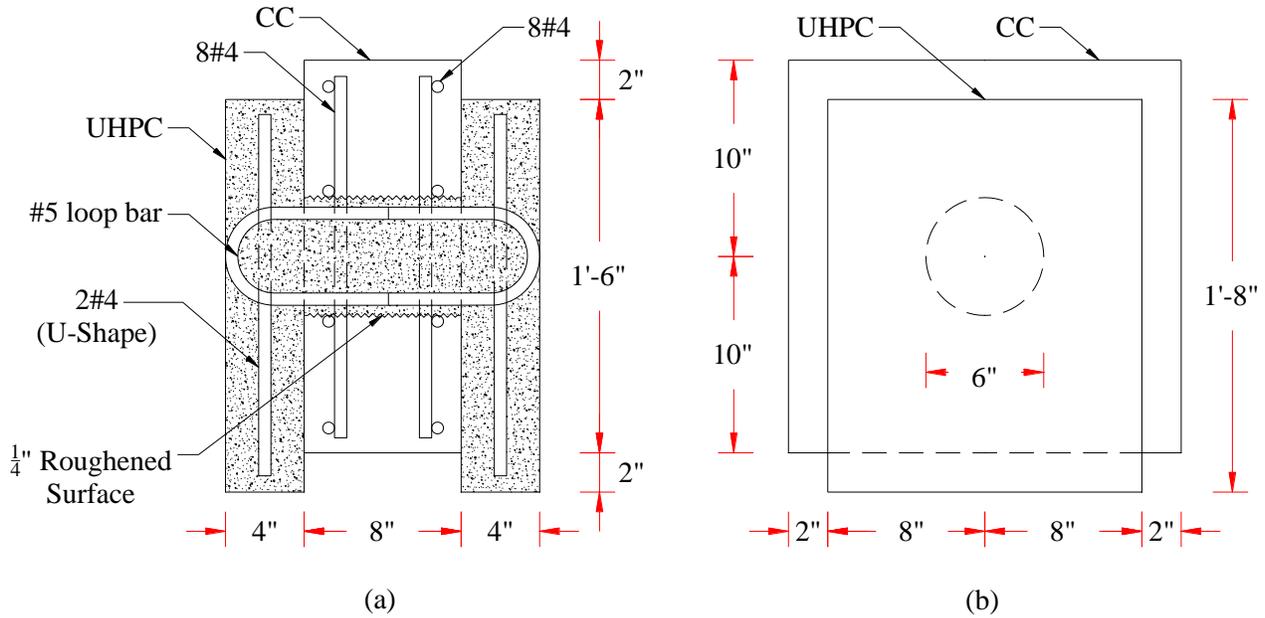


Figure 4.010: Double Shear Push-off Test Specimen Details; (a) Section Elevation, and (b) Side View.

- **Specimen Fabrication**

Figure 4.11 shows the fabrication of the CC slab with 6 in. corrugated plastic pipe. The CC slab was cast and then, was covered with plastic for 28 days to cure. The plastic pipe was removed easily from the shear pocket without damaging the roughening as shown in Figure 4.12(a). In order to eliminate the contact between UHPC and CC, the top and bottom surfaces of CC slabs were covered with wax as shown in Figure 4.12(b). A 2 in. rigid structural foam was used to form the 4 in. UHPC slabs thickness and the No. 5 loop bar was added through the shear pocket as shown in Figure 4.13. The UHPC was cast vertically to fill the slabs and shear pocket. Finally, the top surface of specimens was covered with rigid foam and plastic sheet till the day of testing.



Figure 4.011: Concrete Section of Double Shear Push-off Test Specimen



(a)



(b)

Figure 4.012: Concrete Section Preparation of Double Shear Push-off Test Specimen; (a) Removed Plastic Pipe, and (b) Applying Wax on Concrete Surfaces.

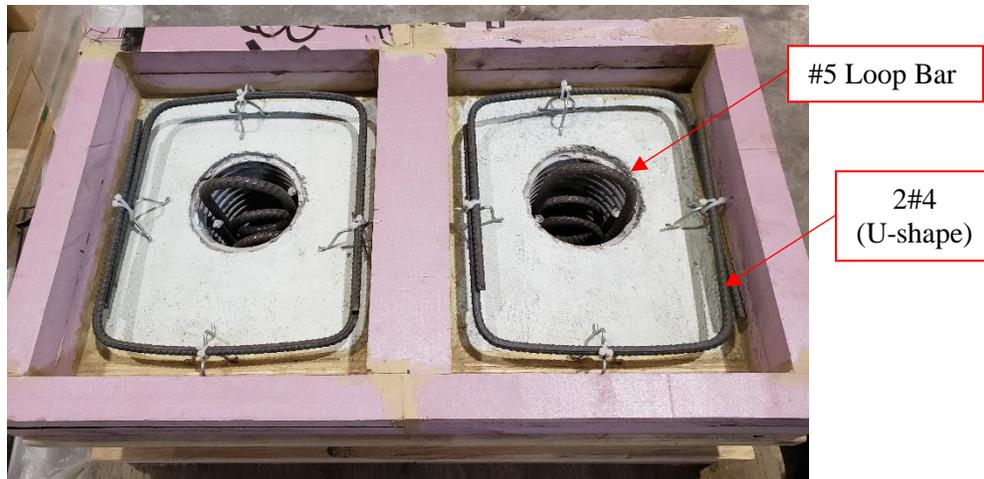


Figure 4.013: Double Shear Push-off Specimen Forming

- **Test Setup and Results**

The precast CC slabs were made using normal weight self-consolidated concrete (SCC) that has a 28-day compressive strength of 6.9 ksi. The compressive strength of UHPC slabs was 17.7 ksi at the day of testing. The specimen base was ground and placed on structural bearing pads to avoid uneven loading. The shear load was applied using a hydraulic ram centered with the specimen till failure. The shear load was distributed over an area of 8 in. x 14 in. using a set of two rigid steel plates to cover the shear pocket area and avoid failure in CC section. Four LVDTs were attached vertically to the specimen, two for each side, to capture the relative vertical displacement between the CC section and the shear pocket center (slippage). Figure 4.13 shows the double shear push-off test setup and instrumentation.

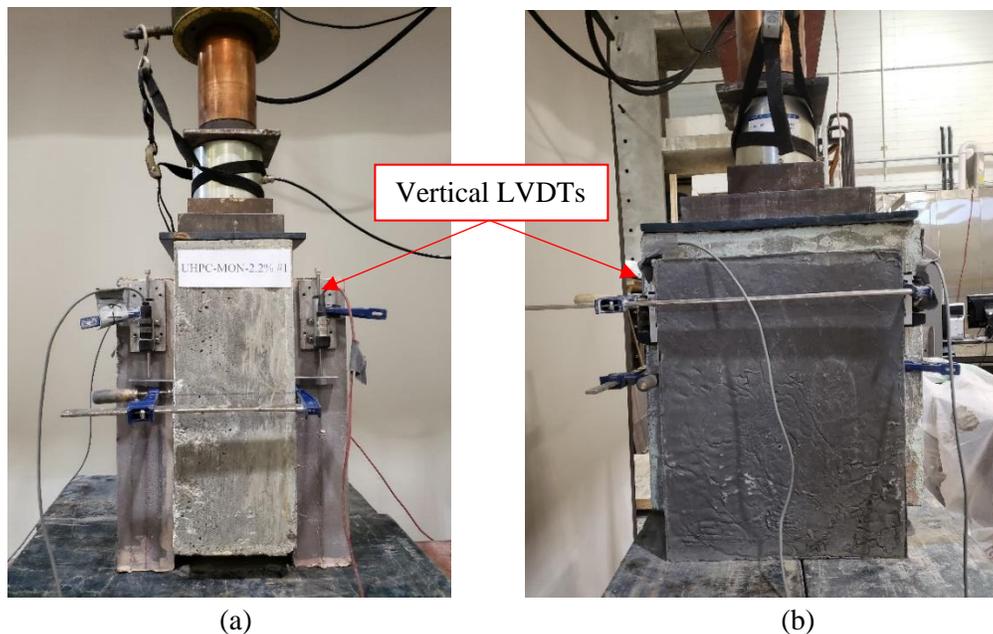


Figure 4.014: Double Shear Push-off Specimen Test Setup; (a) Front View, and (b) Side View.

The two composite specimens exhibited double shear failure at the interface between CC and UHPC as shown in Figure 4.15(a). Figure 4.15(b) shows the rupture of #5 loop bar at the interface plane. The average interface shear resistance is 6.77 ksi for monolithic UHPC with 2.2% interface reinforcement. Figure 4.16 shows the double shear push-off test results. The clamping force produced by interface reinforcement provides more ductility to the interface behavior compared to L-shape specimens without interface reinforcement. The average slippage recorded by the four LVDTs reached 0.1 in. at the peak load which reflects the effect of clamping forces.

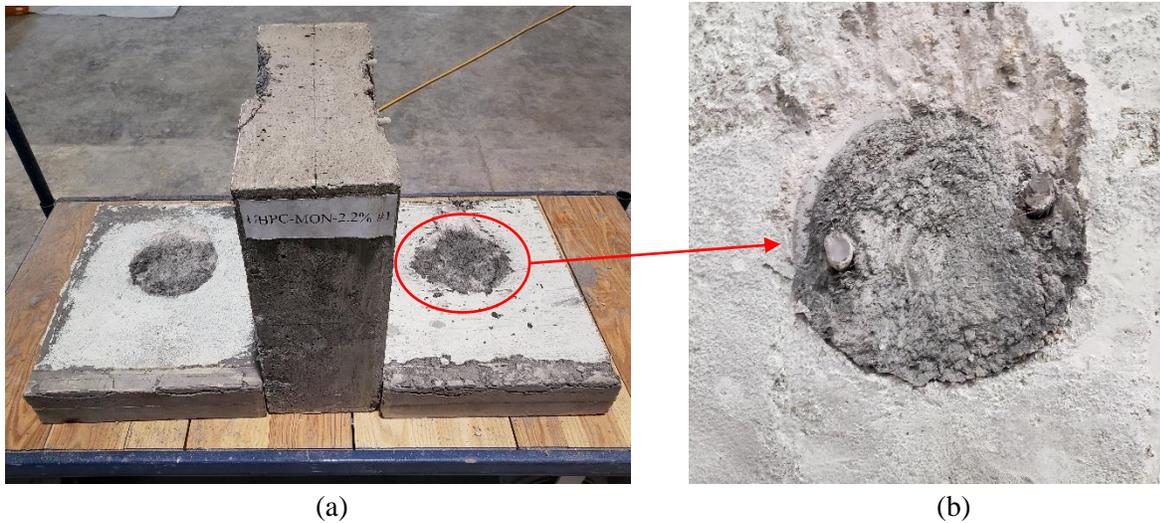


Figure 4.015: Double Shear Push-off Specimen Failure Mode; (a) Double Shear Failure, (b) No. 5 Bar Rupture

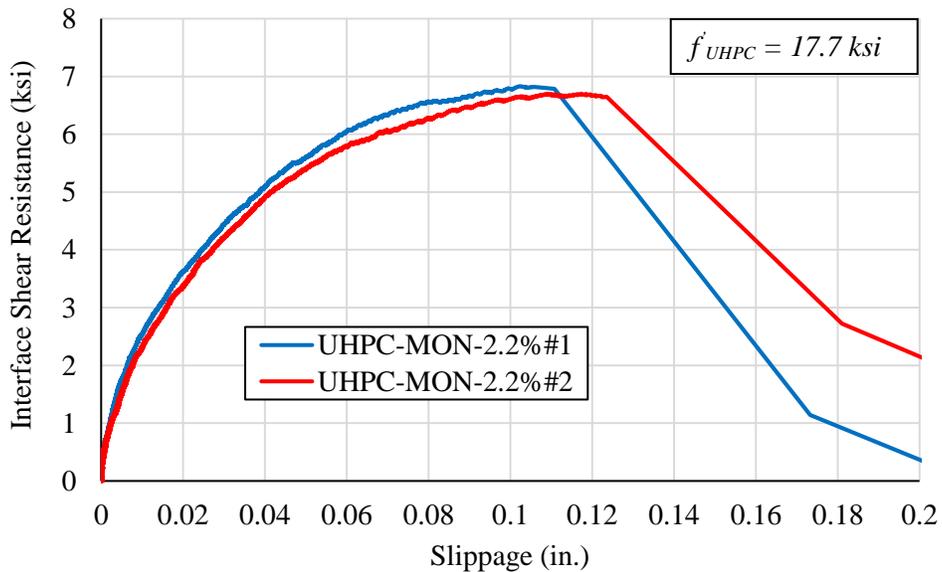


Figure 4.016: Double Shear Push-off Test Results

Crane 2010 performed L-shape push-off test with 0.5% interface reinforcement to obtain the effect of clamping force on the interface resistance. It is worth to mention that the uncracked specimens from Crane 2010 only were considered which has 87 in.² interface area. The cohesion effect was subtracted from the obtained interface resistance using the cohesion factor prediction equation ($c = 0.49\sqrt{f'_{UHPC}}$) to evaluate the effect of clamping force. To eliminate the effect of different UHPC compressive strength, the obtained clamping force was divided by the square root of UHPC compressive strength. According to ACI 318, The yield stress of interface reinforcement is limited to 60 ksi. Table 4.2 shows the interface shear resistance analysis for monolithic UHPC with interface reinforcement. The following equation can be used to predict the interface shear resistance of monolithic UHPC with 2% fiber content:

$$v_{ni} = 0.49\sqrt{f'_{UHPC}} + 0.85\sqrt{f'_{UHPC} * \rho * f_y}$$

Table 4.2: Interface Shear Resistance Analysis of Monolithic UHPC with Interface Reinforcement.

Specimen ID		ρ (%)	v_{ni} (ksi)		f'_{UHPC} (ksi)	$c = 0.49\sqrt{f'_{UHPC}}$ (ksi)	$\mu * \rho * f_y$ (ksi)	μ	$\mu / \sqrt{f'_{UHPC}}$	COV %
Authors	#1	2.2	6.83	6.77	17.7	2.06	4.70	3.56	0.85	1.01
	#2		6.70							
Crane 2010	#1	0.5	3.79	4.02	28.9	2.63	1.39	4.62	0.86	
	#2		4.03							
	#3		4.24							

4.4. Evaluate Interface Shear Resistance of CC-UHPC

The second controlling interface plane is located at the top of concrete girder between fresh UHPC and hardened conventional concrete (CC-UHPC). The interface shear resistance of cc-UHPC was evaluated through slant shear, L-shape push-off tests with and without interface reinforcement as following:

4.4.1. Proposed Equations Based on Literature

Based on AASHTO LRFD 2017 provisions, the nominal interface shear resistance between two CC layers cast monolithically or with cold joint is calculated using the following equation.

$$v_{ni} = c + \mu(\rho f_y + N)$$

Where:

v_{ni} : nominal interface shear resistance (ksi)

c : cohesion factor (ksi)

μ : friction coefficient

ρ : interface shear reinforcement ratio (A_{vf}/A_{cv})

A_{vf} : the area of the interface shear reinforcement crossing the shear plane

A_{cv} : interface shear area

f_y : the reinforcement yield stress which is limited to 60 ksi

N : normal stress applied to the shear plane (ksi)

However, the previous equation can predict the interface shear resistance, it is limited to be used with CC. The data collected from the literature was divided into three categories according to the surface texture: sandblasted, low roughening, and high roughening as shown in Table 4.3. The sandblasted texture category includes only the specimens with sandblasted interface. Low roughening texture category represents the brushed and form liner textures. Grooved, aggregate exposed, and form liners with deep flutes are considered as high roughening category. Different specimens' dimensions and interface angles are used to obtain stresses at interface plane. The interface shear resistance (v_{ni}) and normal stress (N) at the interface plane were calculated by dividing the applied load components based on the interface angle by the interface surface area for each specimen and plotted as shown in Figure 4.17. The interface shear resistance and normal stress are parallel and perpendicular to the interface plane, respectively. The effect of UHPC and CC compressive strength are not significant based on linear regression analysis with interface shear resistance for each of them separately. The parameters of the shear friction model (i.e. cohesion coefficient (c) and friction coefficient (μ)) were obtained for each surface texture and compared against those of AASHTO LRFD for CC with intentionally roughened surface as shown in Table 4.4. The comparison indicated that UHPC cohesion coefficient (c) is significantly higher than that for CC, while UHPC friction coefficient (μ) for low and high roughening surfaces are very close to that of CC. The high coefficients of determination (R^2) of the developed models indicated strong correlations between interface shear resistance and normal stress of CC-UHPC for different surface textures. It worth to mention that the compressive strength of UHPC and CC in Table 4.3 are as mentioned in the literature or an average value if a range was presented. Also, the interface shear resistance values are an average of the mentioned in the literature for the same case.

Table 4.03: Interface Surface Texture Categories Based on the Literature of CC-UHPC Interface Resistance

Surface Texture Category	Reference	Surface Preparation	f'_{cc} (ksi)	f'_{UHPC} (ksi)	Interface Shear Resistance (v_{ni}) (ksi)	Normal Stress (N) (ksi)	Failure Mode
Sandblasted (11)	Muñoz 2012	Sandblasted	8.11	18.35	3.14	1.79	CC
			8.11	18.35	2.12	0.72	CC
	Tayeh et al. 2012		6.50	24.7	2.23	1.29	CC
	Rangaraju et al. 2013		6.77	18.52	3.96	2.28	BF&CC
			6.77	17.92	3.84	2.22	CC
			6.77	22.93	3.58	2.07	CC
			6.77	23	4.10	2.37	CC
Low Roughening (17)	Harris et al 2011	Wire Brushed	5.00	15	1.82	1.05	BF&Mortar
	Muñoz 2012	Brushed	6.46	15.29	2.34	1.36	CC
			6.46	15.29	1.76	0.61	BF
			8.24	12.30	2.59	1.86	CC
			8.11	11.69	2.22	1.22	CC
			8.11	11.69	0.83	0.62	UHPC
	Tayeh et al. 2012	Wire Brushed	6.50	24.7	1.60	0.93	BF&CC
	Aaleti and Sritharan 2017	Form Liner (Panara)	5.20	18.00	2.29	1.72	BF
			7.46	18.00	3.64	2.73	CC
			6.40	18.00	3.16	2.37	BF
		Form Liner (2/61 Thame)	5.20	18.00	1.87	1.40	BF
			7.46	18.00	3.17	2.38	BF
			6.40	18.00	2.59	1.95	BF
		Form Liner (2/98 Vltava)	5.2	18	2.71	2.03	CC
			7.46	18	3.85	2.89	CC
		6.403	18	2.98	2.23	BF	
	High Roughening (16)	Harris et al. 2011	Grooved	5.00	15.00	2.17	1.25
Muñoz 2012		Grooved	6.46	15.29	2.55	1.45	CC
			6.46	15.29	1.63	0.63	CC
			8.11	11.36	1.38	0.45	BF&CC
		Roughened (Aggregate Exposed)	6.61	17.89	2.47	1.36	CC
			6.61	17.89	1.77	0.62	CC
			7.28	12.30	2.43	1.74	CC
			7.28	12.30	1.40	0.56	CC
7.28		11.69	0.49	0.17	BF&UHPC		
Tayeh et al. 2012		Grooved	6.50	24.70	1.75	1.01	BF&CC
Aaleti and Sritharan 2017		Form Liner (Fluted Rib)	5.20	18.00	2.33	1.75	CC
			7.46	18.00	3.77	2.83	CC
			6.40	18.00	3.58	2.69	CC
		Form Liner (2/63 Wisla)	5.20	18.00	2.43	1.82	CC
			7.46	18.00	3.56	2.67	CC
6.40		18.00	4.16	3.12	CC		

*CC: Concrete Failure, BF: Bond Failure, UHPC: UHPC Failure, BF&CC: Bond Failure and Concrete Fracture, BF&UHPC: Bond Failure and UHPC Fracture

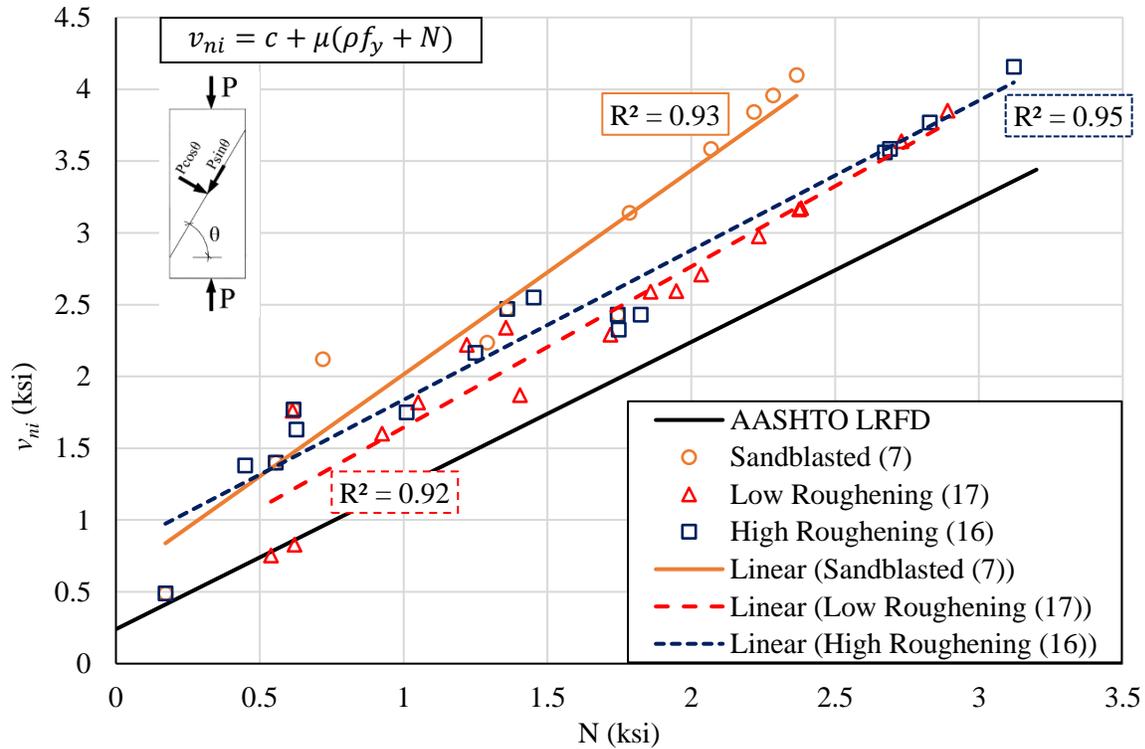


Figure 4.017: Average Interface Shear Resistance of CC-UHPC with Different Surface Textures.

Table 4.4: CC-UHPC Cohesion and Friction Coefficients of Different Interface Surface Textures

Surface Texture	UHPC Cohesion Coefficient (c), ksi	UHPC Friction Coefficients (μ)	R ²
Sandblasted	0.59	1.42	0.93
Low Roughening	0.52	1.12	0.92
High Roughening	0.80	1.0	0.95
AASHTO LRFD	0.24	1.0	NA

4.4.2. Slant Shear Test

A slant shear test based on ASTM C882/C882M was performed to evaluate the interface shear resistance of CC-UHPC. A 4 in. by 8 in. cylinder specimen was used instead of 3 in. by 6 in. to allow the use of conventional concrete as a substrate (Abo El-Khier et al. 2019). Hardened CC cylinders were saw cut diagonally at 60° angle with the horizontal axis. The compressive strength of hardened concrete at 28 days was 8 ksi, which represents the common compressive strength of precast concrete girders. Figure 4.18 shows three different textures applied to interface shear surface using wet saw; as-cut (as cut with the wet saw and without additional treatment), shallow grooved (average 1/8 in. depth), and deep grooved (average 1/4 in. depth).

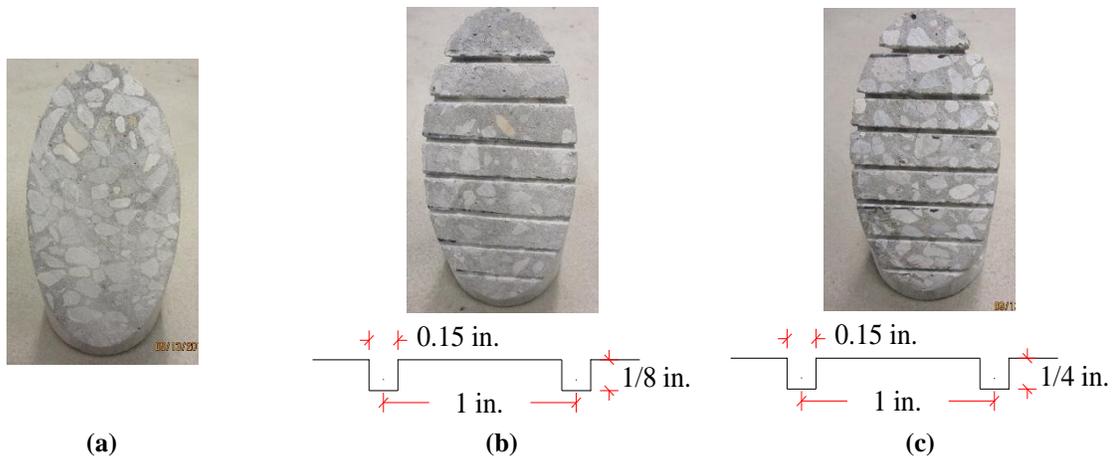


Figure 4.018: Interface Textures of Hardened Concrete Section; (a) Smooth, (b) Shallow Grooved, and (c) Deep Grooved (1in. = 2.54 cm).

The concrete sections were placed back in the molds after applying surface textures and the interface surface was pre-wetted directly before casting UHPC. The composite section specimens were stripped out of the molds after one day and submerged in lime-saturated water in a room temperature of 73°F (23°C) temperature till the day of testing. Both ends of composite section specimen were ground prior to being tested under a compression load rate of 300-400 lb/sec. till failure according to ASTM C39 for CC as shown in Figure 4.19.

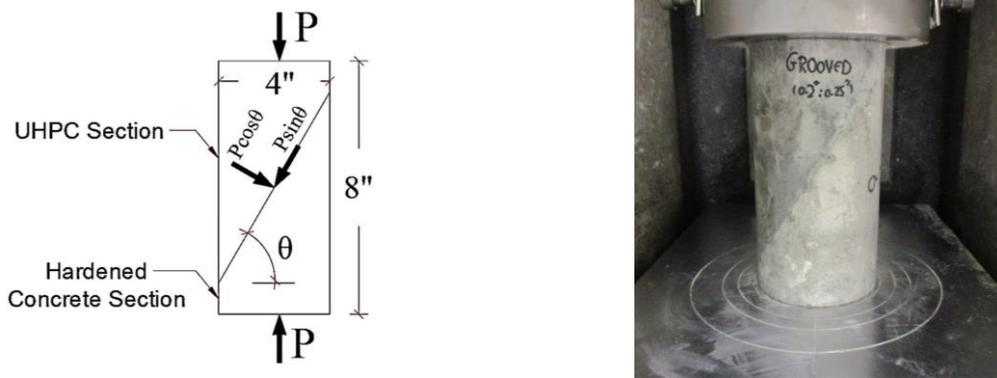


Figure 4.019: Slant Shear Test Specimen Dimensions and Test Setup.

A total of 24 slant shear composite specimens were tested at different UHPC compressive strengths after casting the UHPC section. The CC had an average compressive strength of 8 ksi and UHPC had a compressive strength ranging from 17.67 to 27.2 ksi. Different failure modes were observed for different surface textures as shown in Figure 4.20. All specimens with as-cut surface had bond failure as shown in Figure 4.20(a). All specimens with shallow grooved surface had bond failure with fractured CC as shown in Figure 4.20(b). All specimens with deep grooved surface had failure in the CC portion as shown in Figure

4.20(c). The interface shear resistance and normal stress were calculated by dividing the applied load components, based on the interface angle as shown in Figure 4.19, by the interface surface area (25.1 in.²). Figure 4.21 shows the average interface shear resistance of three identical specimens at different UHPC compressive strengths for each surface texture. This figure indicates that there is no significant difference in the interface shear resistance of CC-UHPC with shallow and deep grooved surface textures. However, different mode failures were exhibited for each surface texture. Also, the interface shear resistance increases with the increase of UHPC compressive strength for as-cut surface texture.

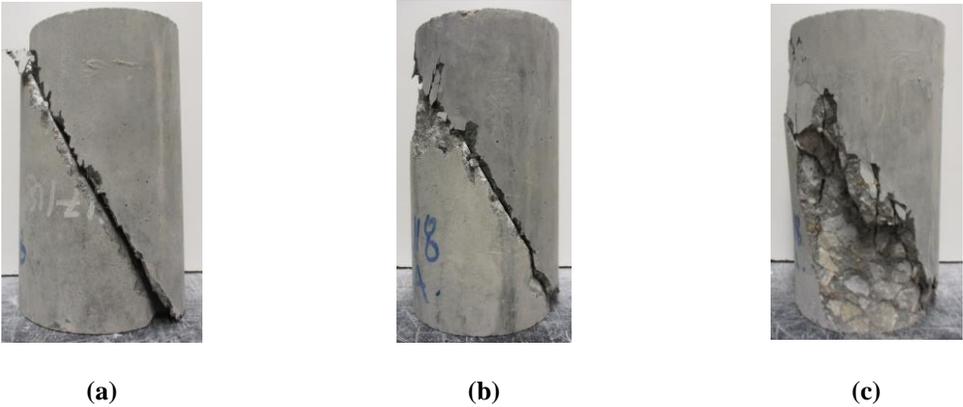


Figure 4.020: Slant Shear Specimen Failure Modes; a) Bond Failure, b) Bond Failure and CC Fracture, and c) CC Failure.

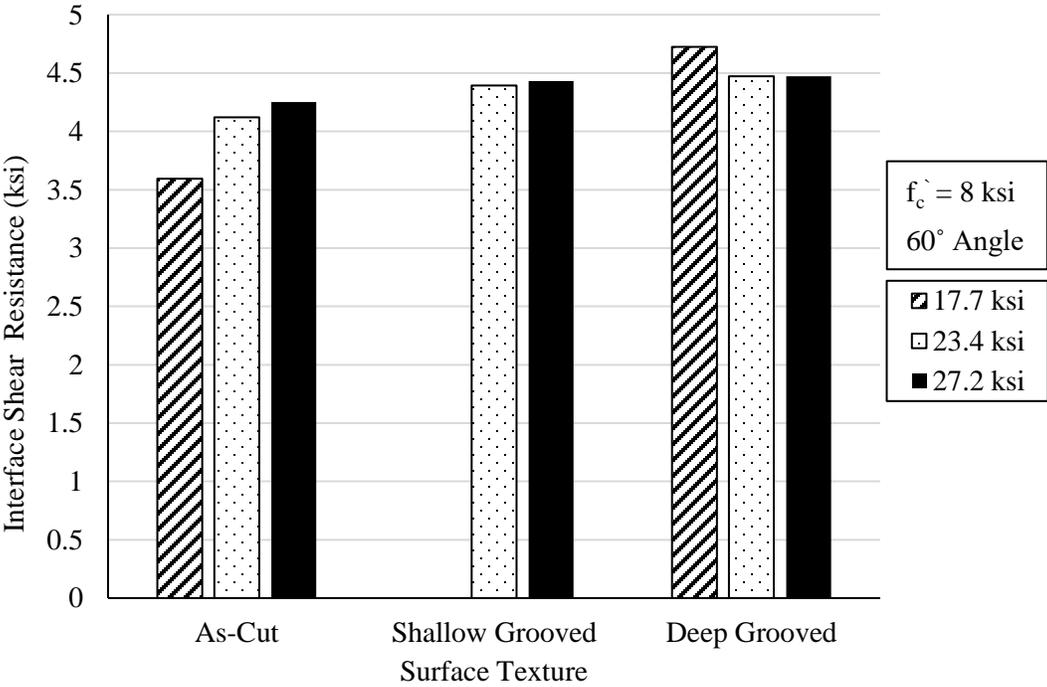


Figure 4.021: Interface Shear Resistance of CC-UHPC at Different UHPC Compressive Strength for Different Surface Textures.

The deep grooved interface surface achieved slightly higher interface shear resistance than as-cut and shallow grooved interface surface for 8 ksi CC and average 24 ksi UHPC. This conclusion might change when different compressive strengths are used. Figure 4.22 plots the slant shear test results against the relations developed from the literature. The plot shows that interface shear resistance of as-cut texture, which have a little roughening due to using wet saw, is very close to predicted values of sandblasted surface texture. The shallow and deep grooved are higher than those of low and high roughening interface due to high compressive of CC. However, the deep grooved surface texture specimens' results validate the proposed equations for high roughening surface texture. The compressive strength of CC section is the upper limit for the interface shear resistance.

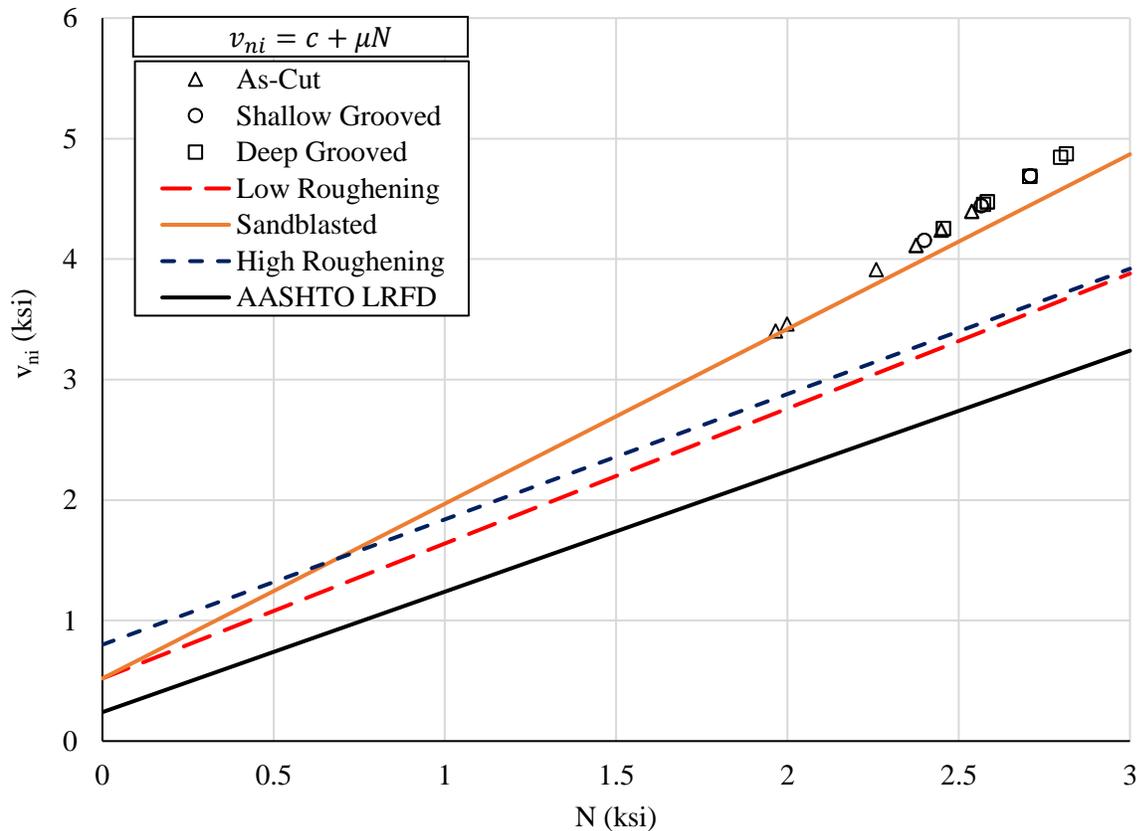


Figure 4.022: Results of Slant Shear Test and their Comparison to the Literature.

4.4.3. L-Shape Push-off Test

L-shape push-off test was conducted to investigate the effect of clamping force produced by interface reinforcement on the interface shear resistance of CC-UHPC. L-shape composite specimen dimension and reinforcement details are shown in 4.23. Three different interface reinforcement were investigated; no reinforcement, 2leg No. 3 stirrup, and 2leg No. 4 stirrup which representing 0%, 0.44%,

and 0.8% reinforcement ratios respectively. The CC section was cast first vertically using a low slump mix to allow applying an interface surface roughening of 1/4 in. depth as shown in Figure 4.24. The CC section stripped out of the form after 24 hours and covered with plastic. The average compressive strength of CC at 28 days was 6.6 ksi that simulate the minimum compressive strength of a concrete girder. The UHPC section was cast vertically to simulate the same practice as a field cast connection. The L-shape composite specimen were stripped out from the form, covered with plastic, and stored in the lab till the day of testing.

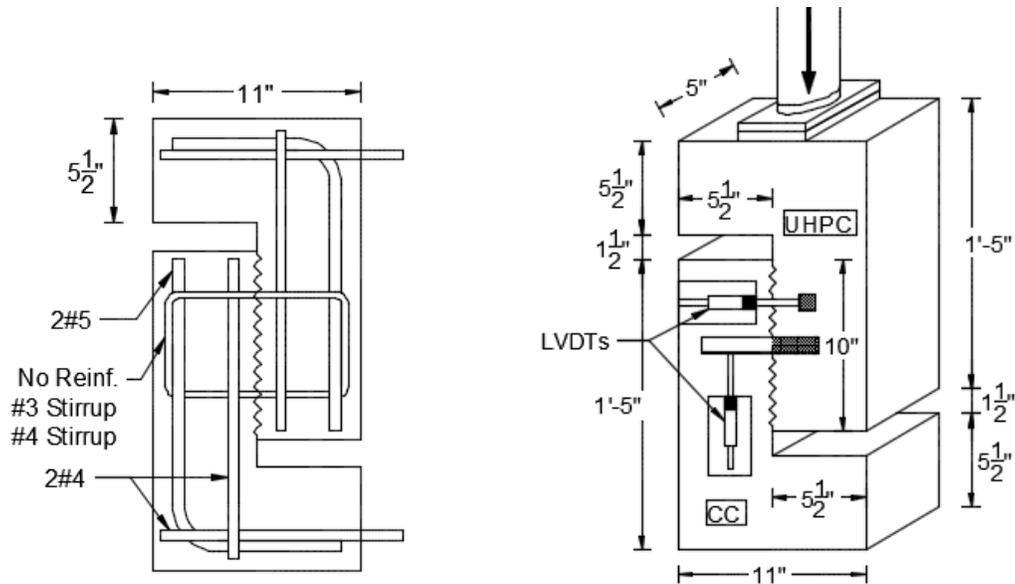


Figure 4.023: L-Shape Push-off Specimen Details and Test Setup.



Figure 4.024: Interface Surface Roughening and Different Reinforcement across Interface; No Reinforcement (Left), 2leg No. 3 Stirrup (Middle), and 2leg No. 4 Stirrup (Right).

Four LVDTs were used to capture the relative horizontal and vertical displacement between the two sections; two LVDTs for each side as shown in Figure 4.25. A shear load rate of 600 lb/sec. was applied till failure using a hydraulic ram after aligning it with the interface plane. The applied load was measured

using a pressure transducer attached to the ram. The specimens were label using the form A-B-C%#D where A is the section cast first, B is the section cast second, C is the interface reinforcement ratio, and D is the specimen number as shown in Table 4.5.

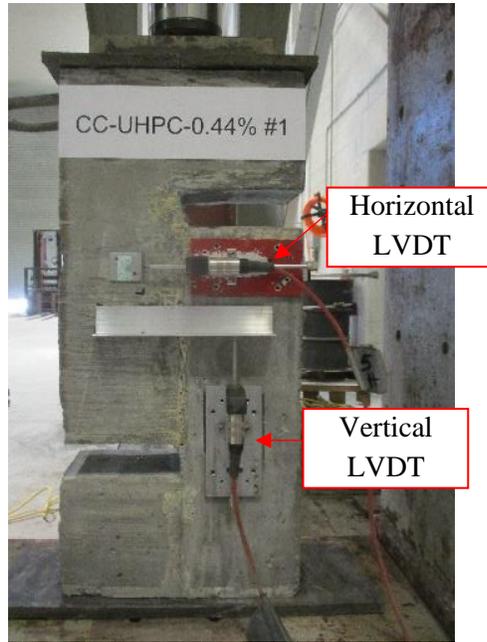


Figure 4.025: L-Shape Push-off Test Setup

Table 4.005: L-Shape Push-off Specimens Details and Labels

Surface Texture	A_{cv} (in. ²)	Interface Reinforcement	A_{vf} (in. ²)	Interface Reinforcement Ratio, $\rho = A_{vf} / A_{cv}$ (%)	Label
Roughened (> 1/4 in. depth)	50.0	None	0.0	0.0	CC-UHPC-0%
		2-Leg No. 3 Stirrup	0.22	0.44	CC-UHPC-0.44%
		2-Leg No. 4 Stirrup	0.4	0.80	CC-UHPC-0.8%

A total of nine L-shape specimens were tested and the maximum applied load was measured as shown in Table 4.6. All the specimens exhibited failure on CC section parallel to the interface plane as a vertical crack at the top reinforcement of CC section as shown in Figure 4.26. Figure 4.27 and Figure 4.28 show the relative vertical displacement (slippage) and relative horizontal displacement at the interface plane of the tested specimens. All the specimens had slippage and relative horizontal displacement less than 0.01 in. at the peak shear load. The CC-UHPC-0% specimens exhibited brittle failure at the peak shear load. However, the interface shear reinforcement provided a clamping force across the interface that enhanced the shear capacity and provided more ductile failure. The L-shape push-off test results show good agreement with the predicted resistance using proposed c and μ factors for high roughening surface texture with depth greater than 0.25 in as shown in Figure 4.29. The results of the L-shape could be explained as a monolithic shear failure on CC section as shown in Figure 4.26.

Table 4.06: L-Shape Push-off Test Results and Compared to Proposed Equation.

Specimen Label	f'_{cc} (ksi)	f'_{UHPC} (ksi)	Maximum Applied Load (kips)	Average Applied Load (kips)	Predicated Load, (kips)	Failure Mode
CC-UHPC-0% #1	6.6	20.47	41.07	42.05	40.0	CC
CC-UHPC-0% #2	6.6	21.03	49.21			CC
CC-UHPC-0% #3	6.6	21.03	35.86			CC
CC-UHPC-0.44% #1	6.6	20.47	67.13	59.88	53.73	CC
CC-UHPC-0.44% #2	6.6	21.03	52.13			CC
CC-UHPC-0.44% #3	6.6	21.03	60.39			CC
CC-UHPC-0.8% #1	6.6	20.47	66.17	63.68	64.0	CC
CC-UHPC-0.8% #2	6.6	21.03	62.95			CC
CC-UHPC-0.8% #3	6.6	21.03	61.94			CC

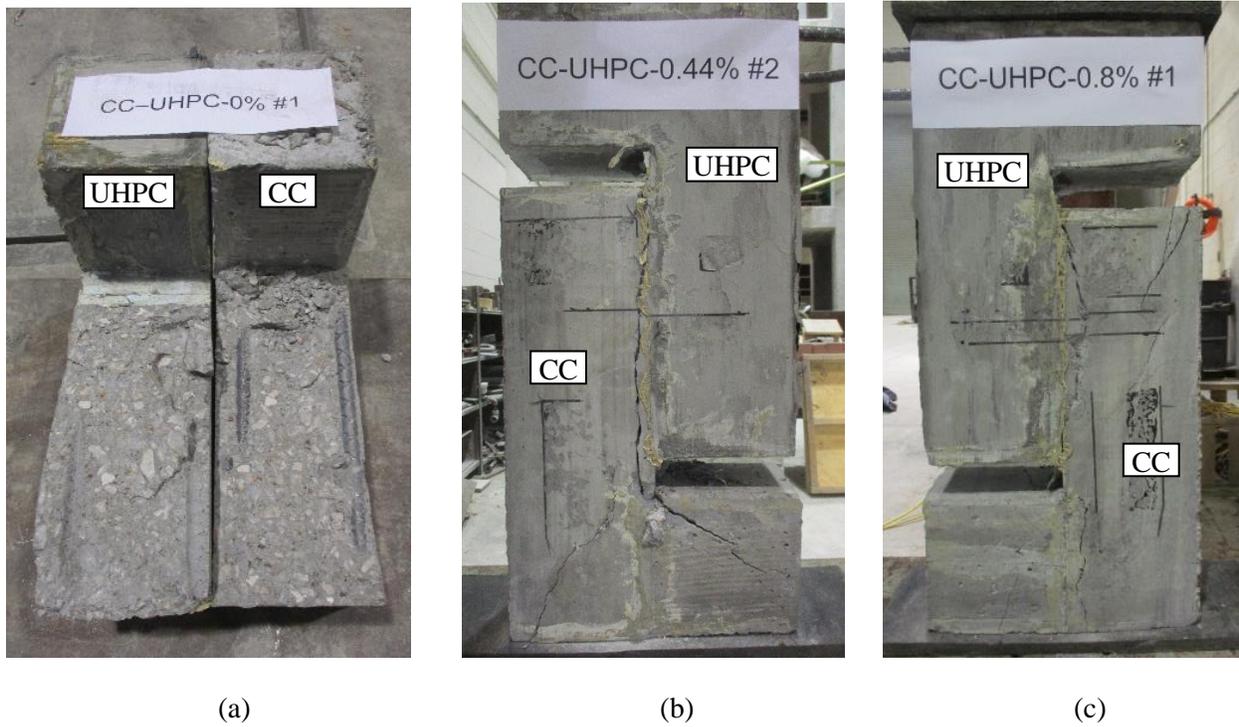
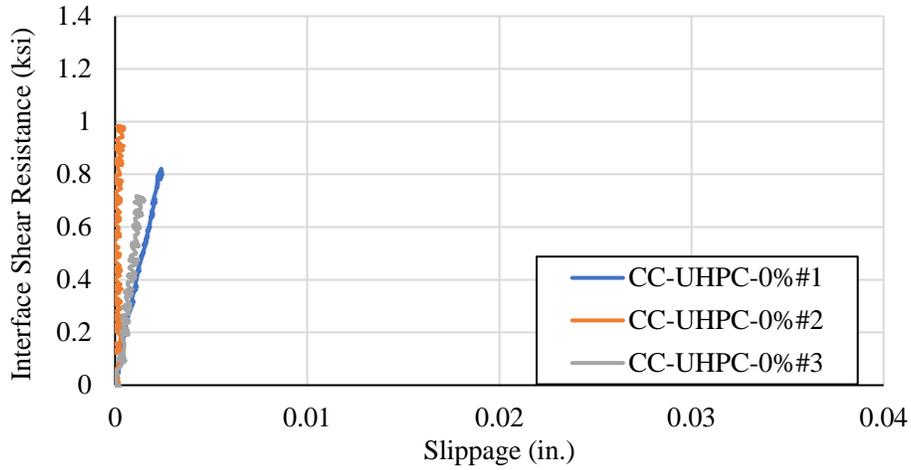
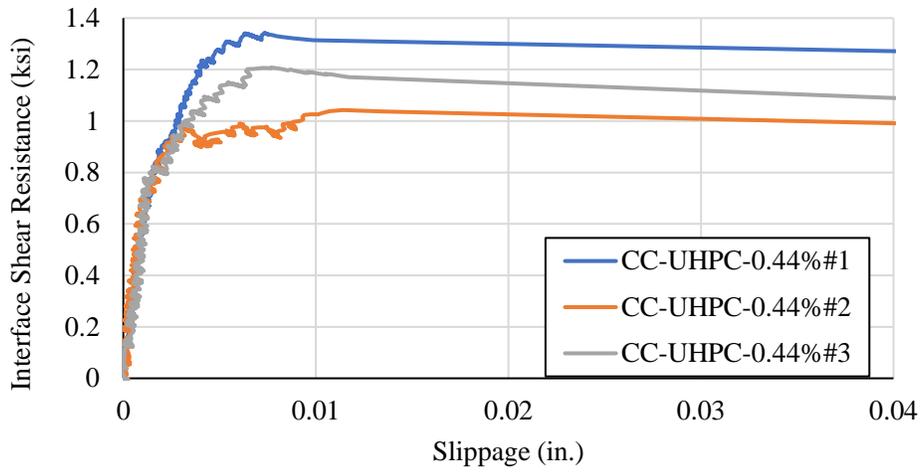


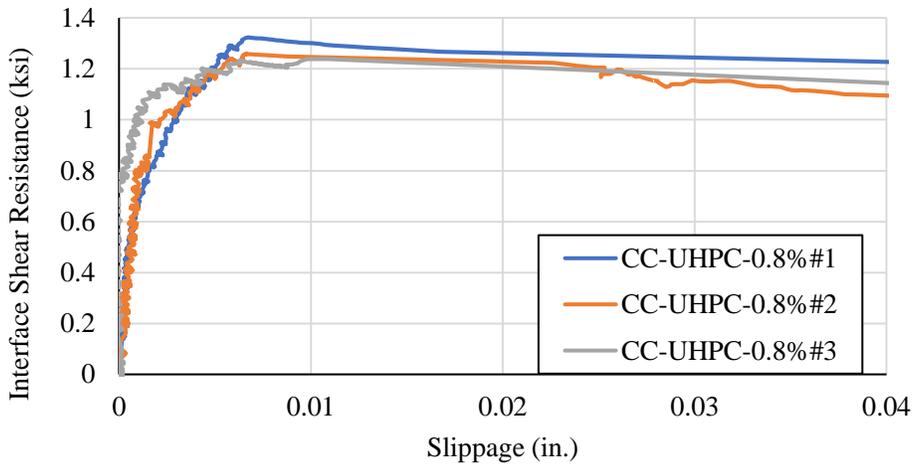
Figure 4.26: CC Failure Mode of L-Shape Specimens with different interface reinforcement ratios; (a) No Reinforcement, (b) 0.44%, and (c) 0.8%.



(a)

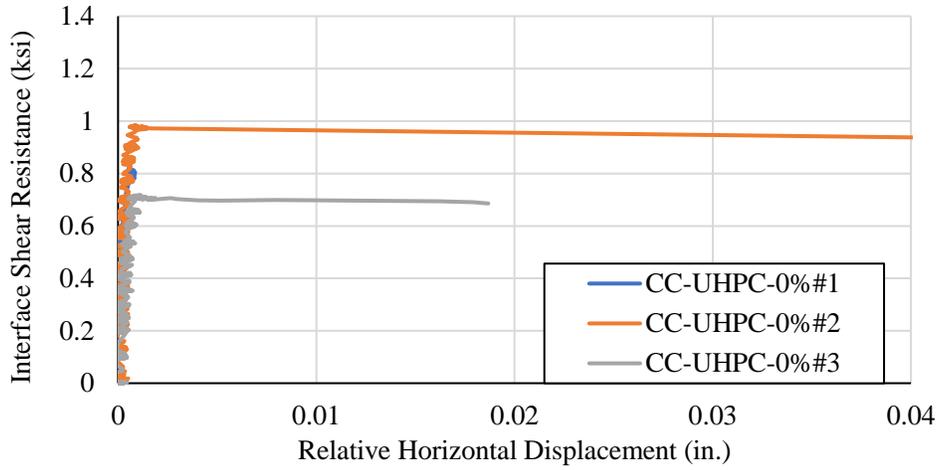


(b)

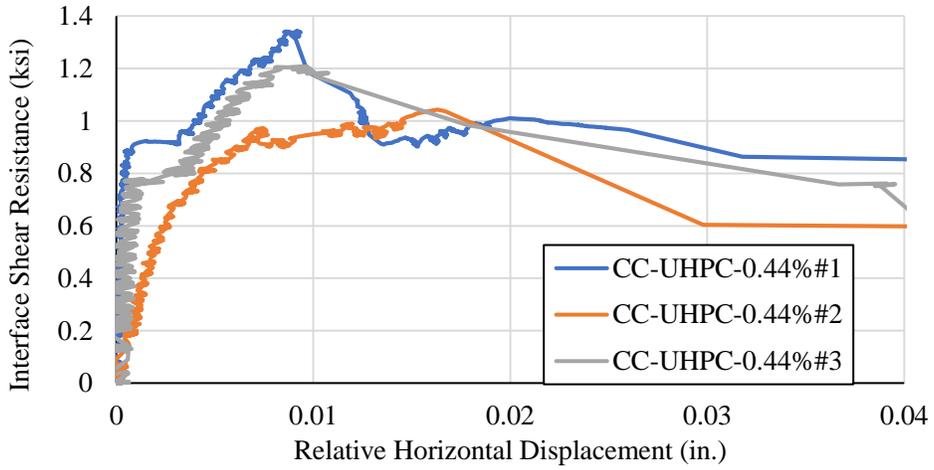


(c)

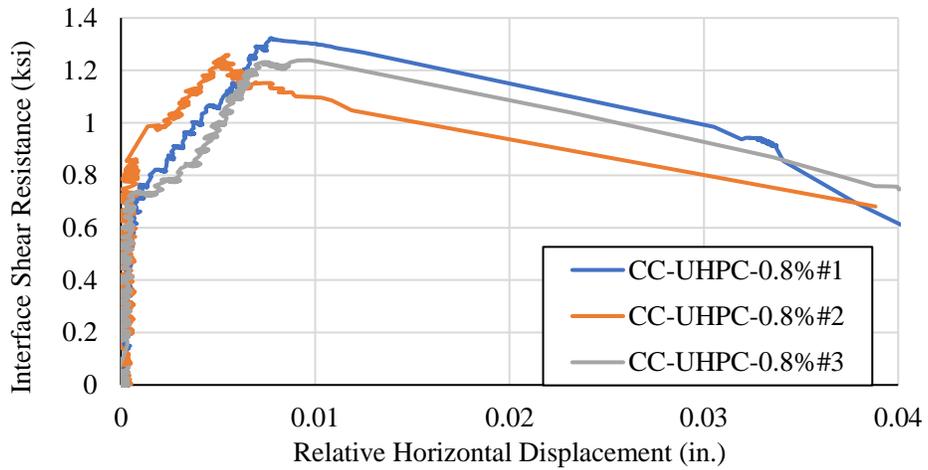
Figure 4.027: Measured Slippage between the Two L-Shape Sections with Different Interface Reinforcement Ratios; (a) No Reinforcement, (b) 0.44%, and (c) 0.8%.



(a)



(b)



(c)

Figure 4.028: Effect of Different Interface Reinforcement on Relative Horizontal Displacement; (a) No Reinforcement, (b) 0.44%, and (c) 0.8%.

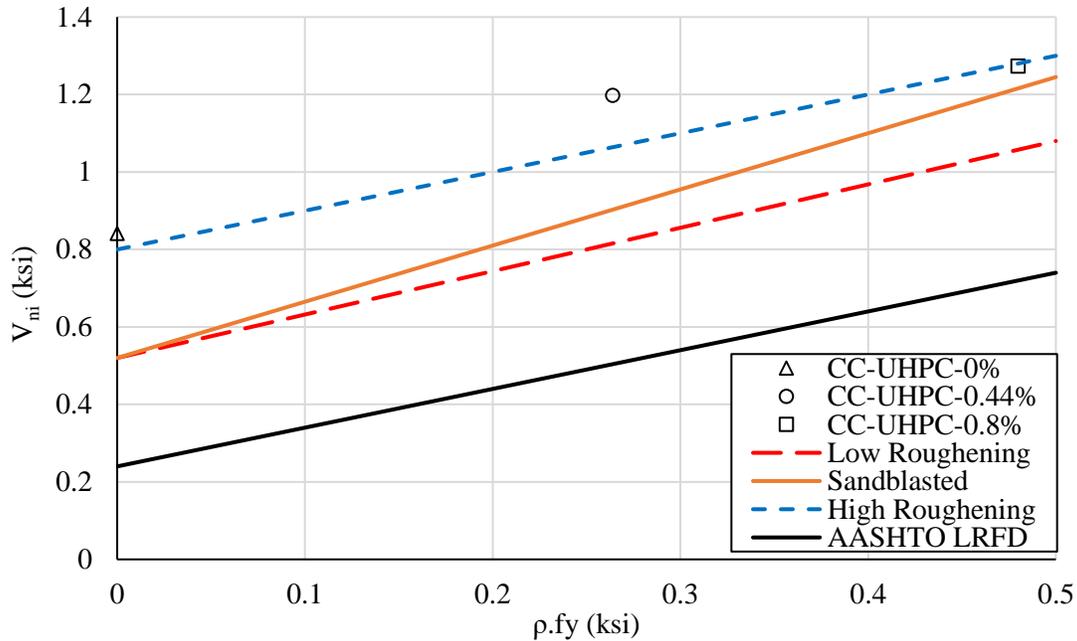


Figure 4.029: Average Interface Resistance of CC-UHPC Obtained from L-Shape Push-off Test and Their Comparison with Proposed Equations.

4.5. Full-Scale Push-off Test

The hidden connection concept with precast bridge elements is a promising construction technique that reduces the number of open joints and the associated grinding requirement. However, casting these connections is a big concern as the cast process is blind to fill the grouting holes and haunch area. Using UHPC, which is sticky, as a grouting material in a blind cast is a challenge that requires to be investigated. Three full-scale push-off specimens were designed and tested to evaluate the structural performance and constructability of the new connection. Concrete blocks were used to simulate the precast/prestressed girder with 16 in. width roughened surface along the center of the girder line. Two No. 4 bars every 2 ft. were used to represent the girder interface shear reinforcement. Two round shear pockets, at 3 ft. spacing, were formed in deck panel using a 6 in. diameter corrugated plastic pipe that provides a roughening along the parameter. Two different UHPC mixes were used to fill the shear pockets and haunch area to achieve the composite action between precast deck panel and concrete girder. A No. 5 loop bar with a length of 10 in. was used to enhance the composite action through providing adequate embedment length in the shear pockets (6 in.) while resting on the girder. Table 4.7 shows the configuration of the full-scale push-off specimens. Figure 4.30 shows the dimensions and reinforcement details of the full-scale push-off specimens.

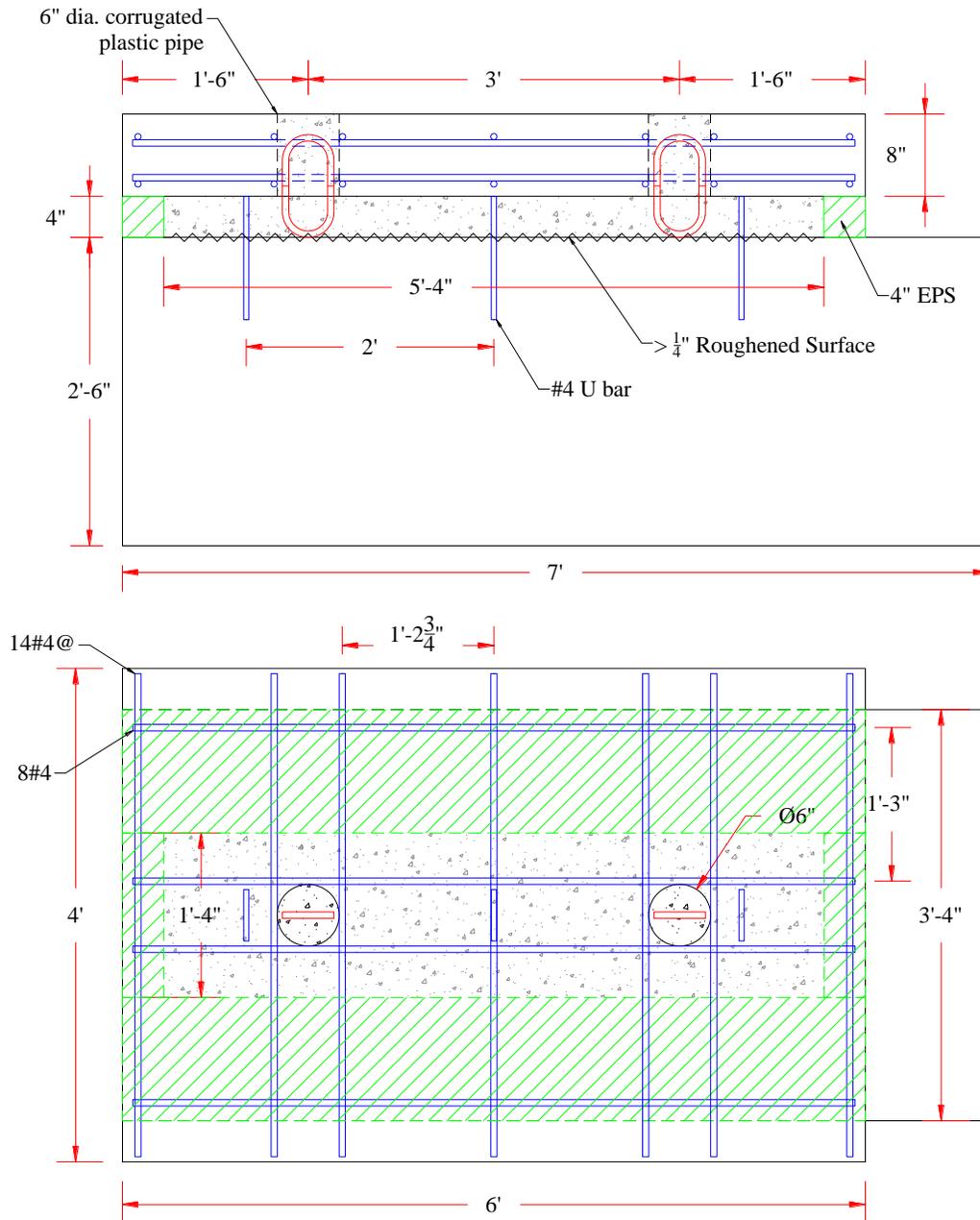


Figure 4.030: Full-Scale Push-Off Specimen Details.

Table 4.7: Full-Scale Push-off Specimens Configuration

Specimen ID	Girder Type	Deck Panel	Shear Pocket
UHPC#1	Concrete Block 3'4" x 7' x 2'6"	Precast Concrete 4' x 6' x 8"	Two 6 in. diameter @ 3 ft. spacing
UHPC#2			
UHPC#3			

- **Specimen Fabrication**

Each shear pocket was formed using a 6 in. diameter corrugated plastic pipes to provide a roughened surface with roughening depth greater than 1/4 in. The plastic pipe is commonly used for drainage systems and it is cheap and commercially available. The bottom and top of the pipe were sealed with liquid nails and plastic sheet, respectively, to prevent the concrete from filling the shear pocket while casting the deck panel. Four bars in two layers were used to hold the plastic pipe in place and strengthen the area around the shear pockets. Figure 4.31 shows the shear pocket forming and deck panel reinforcement details. The deck panel was cast using self-considered concrete (SCC) and then was cured for seven days (covered with plastic sheets and sprayed with water every day) and stored in the laboratory.



Figure 4.031: Shear Pockets Forming and Slab Reinforcement Details.

The concrete blocks, used to simulate the concrete girder, had shear reinforcement every 2 ft. The concrete blocks were cast using SCC and the top surface of the concrete block was intentionally roughened with depth greater than 1/4 in. and width of 16 in. along the block centerline. Then, the blocks were cured the same way as the slabs and stored in the laboratory. Figure 4.32 shows the interface surface at the top of the concrete block and haunch forming with 2 in. rigid foam boards. The deck slab was placed on the formed foam boards and was sealed to prevent any UHPC leakage. A No. 5 loop bar was formed according to the standard hook details with a length of 10 in. The No. 5 loop bar can be either placed before casting the UHPC or after filling half of the shear pocket. Figure 4.33 shows the details of No. 5 loop bar and its installation in the shear pocket. Finally, UHPC was cast to fill the shear pockets and haunch area and then covered the top of the pocket with plywood as shown in Figure 4.34. It worth mentioning that the research

team kept adding UHPC to shear pockets till ensuring filling the pockets to the top surface as shown in Figure 4.35.

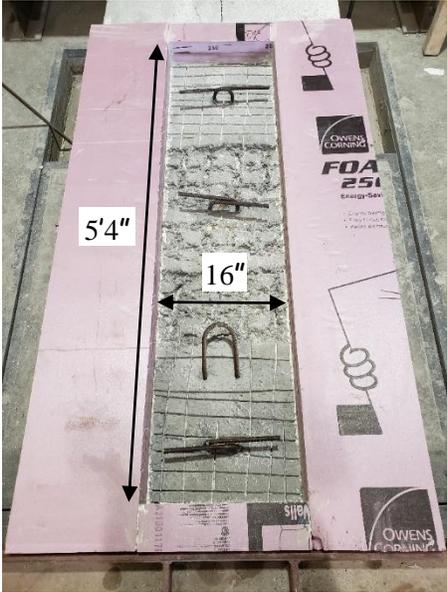


Figure 4.032: CC Interface Shear Area Preparation

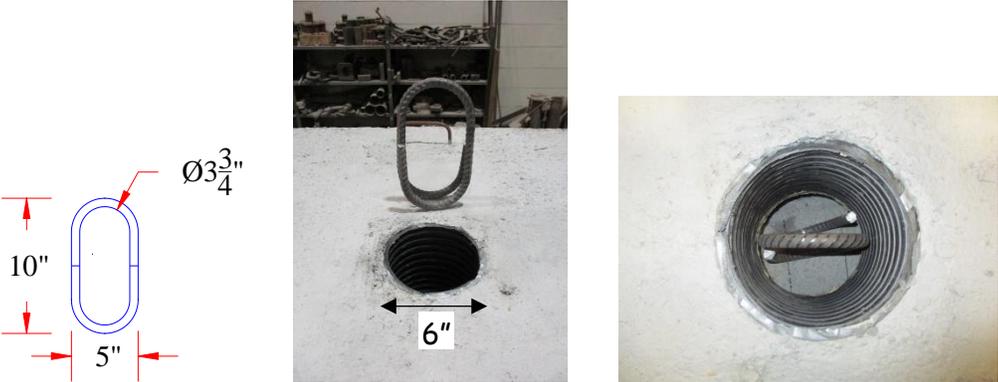


Figure 4.033: No. 5 Loop Bar Details and Installation.



Figure 4.034: UHPC Casting for UHPC#2 Specimen.

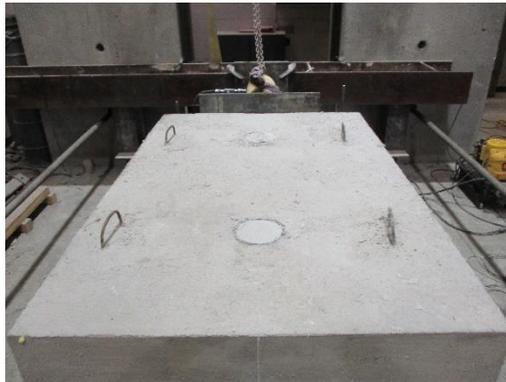


Figure 4.035: UHPC Filled Shear Pockets to Top Surface.

- **Material Properties**

The precast concrete deck panels were made using normal weight self-consolidated concrete (SCC) that has a 28-day compressive strength of 6.6 ksi for UHPC#1 and 7.34 ksi for the other two specimens. The push-off girder concrete was cast using a ready-mixed SCC with an average slump flow of 22 in and average 28-day compressive strength of 6.8 ksi. For each connection, a total of 3.2 cubic feet of UHPC was grouted to fill the two shear pockets and haunch area. The flowability of UHPC batches were measured, at a temperature of 80° F and relative humidity of 50%, using 10 in. diameter flow table according to ASTM C230, specified by ASTM C1856. The three UHPC batches had a flow of 9.5 in., > 10 in., and >> 10 in. for UHPC#1, UHPC#2, and UHPC#3 specimens, respectively. Figure 4.36 shows cross-sections of

hardened 3"x6" cylinders, cut in half longitudinally using a wet saw, for each of UHPC mixes. The UHPC#1 cylinder cross-section had a good distribution of fiber along the section. Also, UHPC#2 cylinder had good fiber distribution with a minor segregation that can be noticed at the top part of the section. However, the third cross-section had a severe fiber segregation that barely had fibers at the top half of the cylinder. These three scenarios were done intentionally to study the effect of flowability on the performance of proposed connection. The three full-scale push-off specimens were tested at an average UHPC compressive strength of 18 ksi.

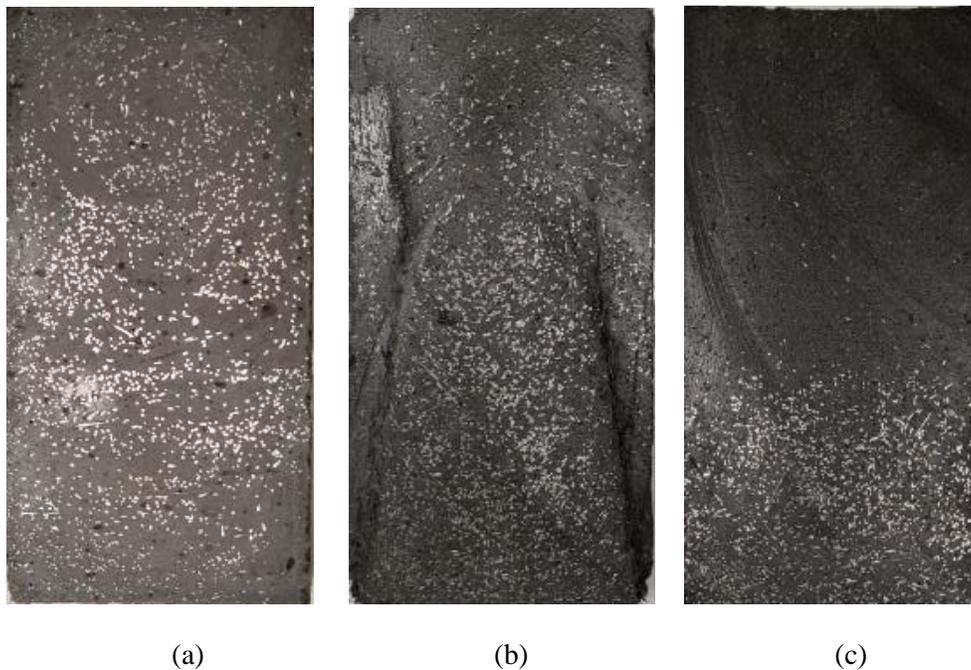


Figure 4.036: Cross-Section of UHPC Cylinders Obtained from Each Full-Scale Push-Off Specimen; (a) UHPC#1, (b) UHPC#2. And (c) UHPC#3

- **Test Setup and Results**

After the UHPC achieved a compressive strength of 18 ksi, the full-scale push-off specimens were tested by applying horizontal load to the deck slab using a hydraulic ram as shown in Figure 4.37. A set of steel plates was used to distribute the applied load over a larger area gradually. The concrete blocks were anchored to the floor through a set of two beams and two threaded rods to prevent the vertical moving of the specimen while applying load. Also, the concrete block was prevented from horizontal movement through placing a steel beam at the end of the block that was fixed by two threaded rods to a concrete retaining wall. Four LVDTs were used to capture the relative vertical and horizontal displacement between the deck panel and concrete block at the first shear pocket; two LVDTs for each direction. The load was measured using a pressure transducer attached to the ram till failure.

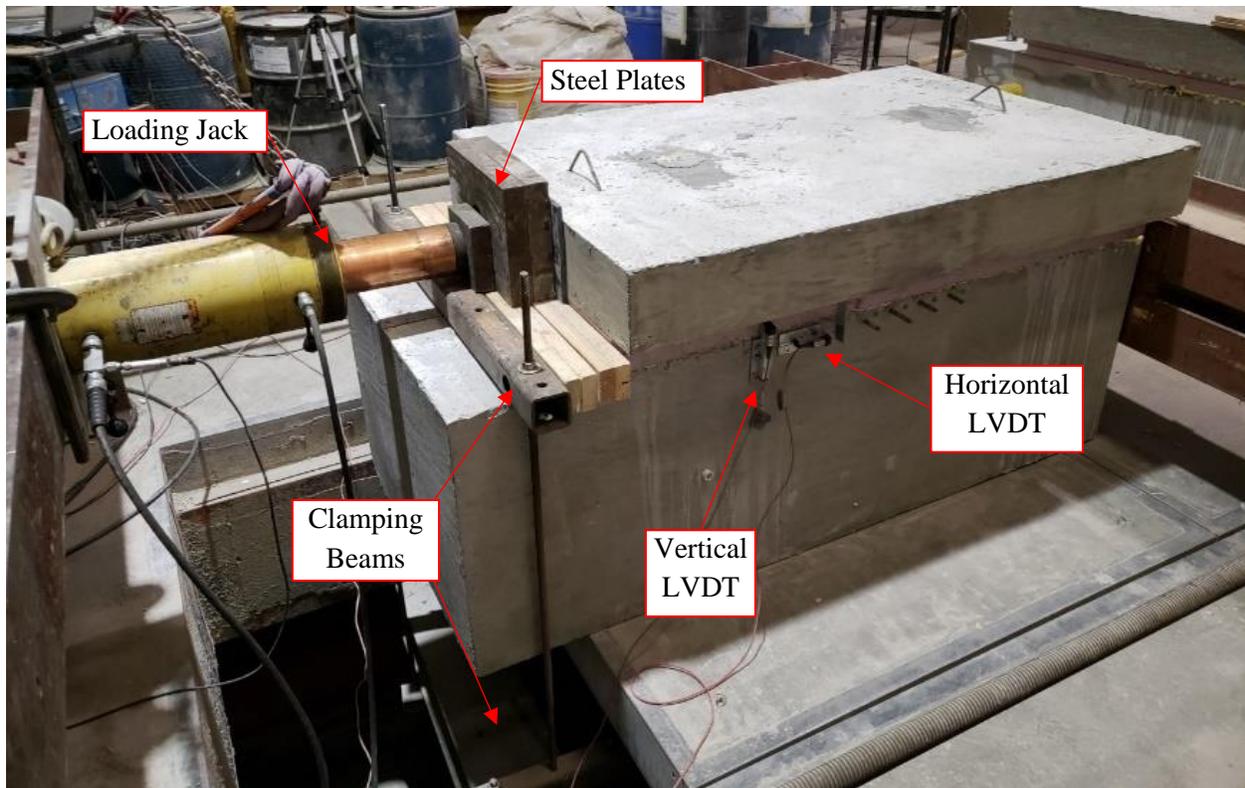


Figure 4.037: Full-Scale Push-Off Specimen Test Setup.

Table 4.9 shows the maximum applied load and the corresponding UHPC compressive strength. Figure 4.38 and Figure 4.39 show the relative vertical and relative horizontal (slippage) displacements of full-scale specimens. However, the three specimens followed the displacement curve, different failure modes were exhibited. UHPC#1 specimen exhibited interface shear failure at the first pocket and CC-UHPC at the end of the block as shown in Figure 4.40(a). This CC-UHPC failure can be attributed to the lack of interface reinforcement at the specimen end which is the same case as real girder; more shear reinforcement at the end. The other two specimens had no interface failure and the loop bar were pulled out from the UHPC haunch as shown in Figure 4.40(b) and (c). The fiber segregation in these two specimens affected the bond strength between the UHPC and embedded loop bar on contrary with UHPC#1 specimen. The large-scale push-off test gives a lower boundary for the interface shear resistance of the proposed connection represented in UHPC#1 specimen as the real bridge has a series of pockets resisting the horizontal force. Also, The effect of load eccentricity on the shear pockets and the pull-out force accumulated with the horizontal applied force affect the results of the large-scale push-off test. However, the test showed that the provided side surface roughening of shear pocket was adequate to prevent pocket pull-out and instead pull-out the embedded bar. The effect of UHPC mix stability can be observed in UHPC#3 results as its UHPC mix had the highest flowability that causes fiber segregation even with higher compressive strength than UHPC#2. The stability of UHPC mix is a key parameter that highly impact the

capacity of the proposed connection. The field-cast UHPC must have less than 10 in. flowability to achieve the full capacity of the proposed connection.

Table 4.08: Full-Scale Push-off Test Results.

Specimen ID	f'_{UHPC} (ksi)	Maximum Load (kips)	V_{ni} (kips/in.)
UHPC#1	18.40	305	4.24
UHPC#2	17.36	240	3.33
UHPC#3	18.40	192	2.67

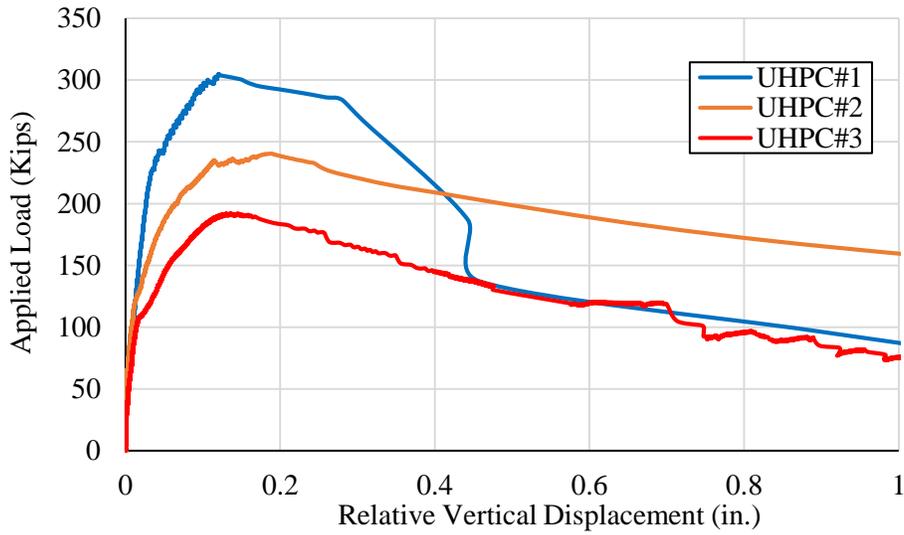


Figure 4.38: Relative Vertical Displacement of Full-Scale Push-off Specimens.

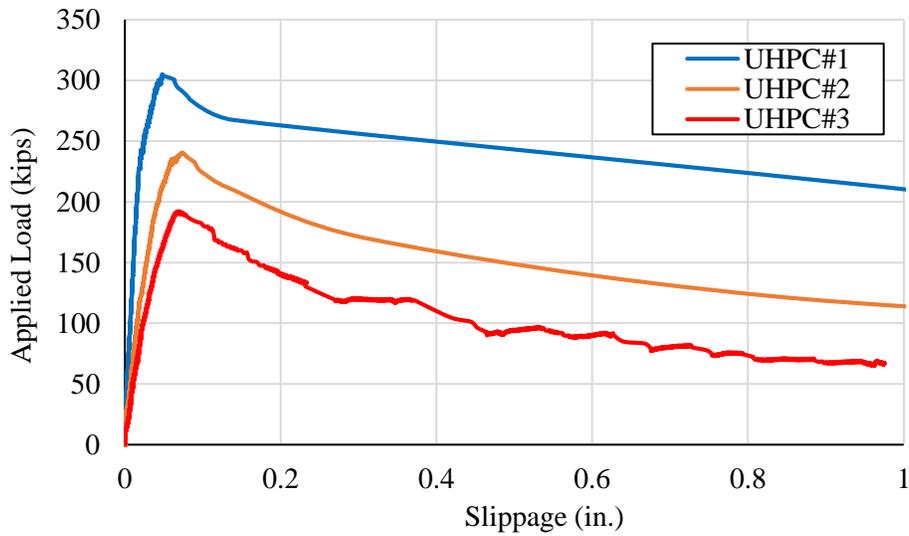


Figure 4.039: The measured Slippage of Full-Scale Push-off Specimens.



(a)



(b)



(c)

Figure 4.040: Full-Scale Specimen Failure Modes; (a)UHPC#1, (b)UHPC#2, and (c)UHPC#3.

Chapter 5. Design Procedure

5.1. Introduction

This chapter provides a design methodology for the proposed connection based on the prediction equations obtained from experimental investigations. An example bridge from PCI Bridge Design Manual 2014 (PCI BDM Ex. 9.1a) is used to present the design procedure of the proposed connection. Design aid charts were generated to simplify the connection design.

5.2. Design Procedure

The advantages of utilizing the proposed connection in deck-to-girder systems are two folds. First, the exceptional properties of UHPC allow eliminating any changes in the design and producing of precast/prestressed concrete girder. Second, UHPC provides high monolithic and CC-UHPC interface shear resistance that simplifies the design procedures (no need for HSS shear pockets or special treaded connectors). However, the design codes do not provide any provisions for calculating the nominal capacities. So, the experimental section of this proposal provides cohesion and friction coefficients that is implemented in the design procedure. Figure 4.25 shows a flowchart that illustrates a general design procedure for the proposed system.

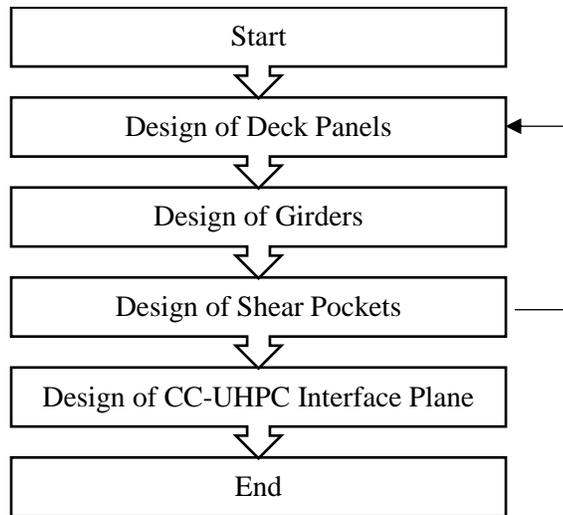


Figure 5.1: Flowchart of General Design Procedures for Proposed System.

Four main parameters controlling the design of proposed connection that a criterion is proposed to determine. Two different failure modes are expected, monolithic interface failure, CC-UHPC interface failure. The design procedure starts with obtaining the nominal interface shear resistant demand from the bridge load cases. Then, the spacing between shear pockets is calculated based on preliminary pocket

diameter and loop bar size. The spacing between shear pockets is recommended to be from 2 to 4 ft. The obtained spacing is used to predict the minimum UHPC haunch width using the girder shear reinforcement; obtained from the girder shear design. If the minimum UHPC haunch width is greater than the girder top flange width, the interface shear reinforcement needs to be designed based on considering the top flange width as UHPC haunch width. The design procedure is shown in Figure 5.2 and provided on a Mathcad file to obtain the following outcomes:

- Shear pocket diameter and spacing
- Loop bar embedded in the shear pocket
- Girder shear reinforcement
- Haunch width

Where:

A_{cv-MN} : monolithic UHPC interface shear area (in.²)

A_{cv-CC} : CC-UHPC interface shear area (in.²)

A_{vf-MN} : interface shear reinforcement Area across monolithic UHPC plane (in.²)

A_{vf-CC} : interface shear reinforcement Area across CC-UHPC plane (in.²)

A_{sl} : loop bar area (in.²)

A_{s2} : girder shear reinforcement bar area (in.²)

b : girder top flange width (in.)

b_w : UHPC haunch width (in.)

c_{MN} : monolithic UHPC cohesion coefficient (ksi)

c_{cc} : CC-UHPC cohesion coefficient (ksi)

D_p : shear pocket diameter (in.)

f'_c : compressive strength of precast deck slab panel (ksi)

f'_{UHPC} : compressive strength of field cast UHPC (ksi)

f_{yh} : bar yield strength (ksi)

N : Number of interface reinforcement bar legs crossing interface plane

V_{ni} : nominal interface shear resistance per unit length (kips/in.)

V_{ni-MN} : nominal interface shear resistance of monolithic UHPC plane (kips)

V_{ni-CC} : nominal interface shear resistance of CC-UHPC plane (kips)

S : girder shear reinforcement spacing (in.)

S_p : spacing between shear pockets (in.)

μ_{MN} : monolithic UHPC friction coefficient

μ_{cc} : CC-UHPC friction coefficient

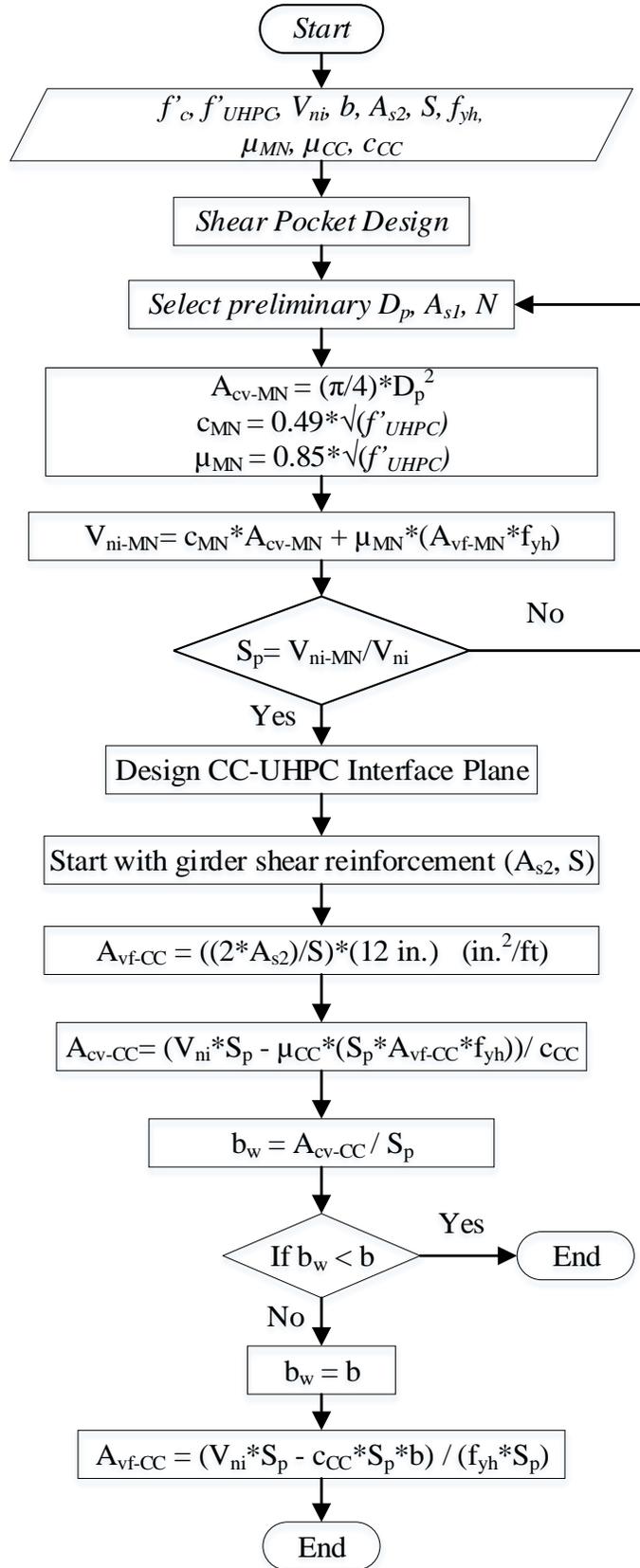


Figure 5.2: Design Procedure flowchart of new connection.

An example bridge from PCI Bridge Design Manual 2014 (PCI BDM Ex. 9.1a) is used to present the design procedure of the proposed connection. In this example, the interface shear resistance prediction equations of monolithic UHPC and CC-UHPC obtained from the experimental investigation are used to design the connection that satisfy the interface shear demand. The design procedure of PCI BDM Ex. 9.1a is shown as following:

Interface Shear Resistance Design for PCI BDM Ex. 9.1a using Proposed UHPC Deck-to-Girder Connection

Deck Panel Concrete Compressive Strength $f'_c := 6 \text{ ksi}$

UHPC Haunch Compressive Strength $f'_{UHPC} := 18 \text{ ksi}$

According to PCI Bridge Design Manual Example 9.1a Section 9.1a.12:

Factored Interface Shear due to DW and LL at h/2 (DC does not apply to the composite section) $V_u := 1.5 \cdot 10.8 \text{ kip} + 1.75 \cdot 104.4 \text{ kip} = 198.9 \text{ kip}$

Shear Depth $d_v := 75.78 \text{ in} - \frac{7.5 \text{ in}}{2} = 72.03 \text{ in}$

Ultimate Interface Shear at Critical Section $V_{ui} := \frac{V_u}{d_v} = 2.76 \frac{\text{kip}}{\text{in}}$

Strength Reduction Factor $\phi := 0.9$

Nominal Interface Shear Resistance per unit length $V_{ni} := \frac{V_{ui}}{\phi} = 3.07 \frac{\text{kip}}{\text{in}}$

UHPC Monolithic Interface Shear Resistance

Pocket Diameter $D_p := 6 \text{ in}$ (4 to 8 in.)

Interface Shear Area Per Pocket $A_{cv_MN} := \frac{\pi}{4} \cdot D_p^2 = 28.27 \text{ in}^2$

Monolithic Cohesion Coefficient $c_{MN} := 0.49 \cdot \sqrt{f'_{UHPC} \cdot \text{ksi}} = 2.08 \text{ ksi}$

Monolithic Friction Coefficient $\mu_{MN} := 0.85 \cdot \frac{\sqrt{f'_{UHPC} \cdot \text{ksi}}}{\text{ksi}} = 3.61$

Embedded Loop Bar Area (Single Leg) $A_{s1} := 0.31 \text{ in}^2$

Number of Loop Bar Legs crossing Interface $N := 2$

Monolithic Interface Shear Reinforcement $A_{vf_MN} := N \cdot A_{s1} = 0.62 \text{ in}^2$

Yield Strength	$f_{yh} := 60 \text{ ksi}$
Nominal Interface Shear Resistance Per Pocket	$V_{ni_MN} := c_{MN} \cdot A_{cv_MN} + \mu_{MN} \cdot (A_{vf_MN} \cdot f_{yh}) = 192.9 \text{ kip}$
Spacing Between Pockets	$S_p := \frac{V_{ni_MN}}{V_{ni}} = 5.24 \text{ ft}$
	$S_p := 4 \text{ ft} \quad (2 \text{ to } 4 \text{ ft.})$

Conventional Concrete (CC)-UHPC Interface Shear Resistance

Width of Girder Top Flange	$b := 48 \text{ in}$	
Girder Shear Reinforcement Bar	$A_{s2} := 0.2 \text{ in}^2$	Use the same shear reinforcement obtained from girder shear design
Girder Shear Reinforcement Spacing	$S := 12 \text{ in}$	
CC-UHPC Interface Shear Reinforcement	$A_{vf_CC} := \frac{2 \cdot A_{s2}}{S} \frac{12 \text{ in}}{1 \text{ ft}} = 0.4 \frac{\text{in}^2}{\text{ft}}$	
CC-UHPC Cohesion Coefficient	$c_{CC} := 0.8 \text{ ksi}$	
CC-UHPC Friction Coefficient	$\mu_{CC} := 1.0$	
Yield Strength	$f_{yh} := 60 \text{ ksi}$	
CC-UHPC interface area	$A_{cv_CC} := \frac{V_{ni} \cdot S_p - \mu_{CC} \cdot (S_p \cdot A_{vf_CC} \cdot f_{yh})}{c_{CC}} = 0.4 \text{ ft}^2$	
Minimum Roughened width of Girder Top Flange	$b_{w_min} := \min\left(\frac{A_{cv_CC}}{S_p}, b\right) = 1.335 \text{ in}$	

5.3. Design Aid

Design aid charts were generated using the proposed equation for predicting the interface shear resistance of monolithic UHPC. The interface shear resistance of shear pockets was calculated for shear pocket spacing ranging from 2 – 4 ft. with different embedded loop bar size. The combination between pocket diameter and loop bar size was chosen to satisfy the standard hook details in accordance with ACI 318. Figure 5.3 and Figure 5.4 show the generated design aids presenting 17 ksi and 21.7 ksi compressive strength of UHPC, respectively. The design aid chart legend is labeled using the form DA#B where D is diameter, A is the shear pocket diameter (in.), and B is the embedded loop bar size.

The design aid charts simplify the design procedure of the new connection. The example bridge from PCI Bridge Design Manual 2014 (PCI BDM Ex. 9.1a) has nominal interface shear demand of 3.07 kips/in. Figure 5.5 demonstrate the utilization of the design aid chart for UHPC of 17 ksi compressive strength to satisfy the demand. Table 5.1 shows the design alternatives for shear pocket diameter and embedded loop bar size based on design chart. These design alternatives satisfy the interface demand which is at the maximum shear force. The interface shear demand is significantly reduced towards the middle of the girder. The authors recommend choosing the design alternative that will allow using the same design all over the girder to simplify the panel production and erection.

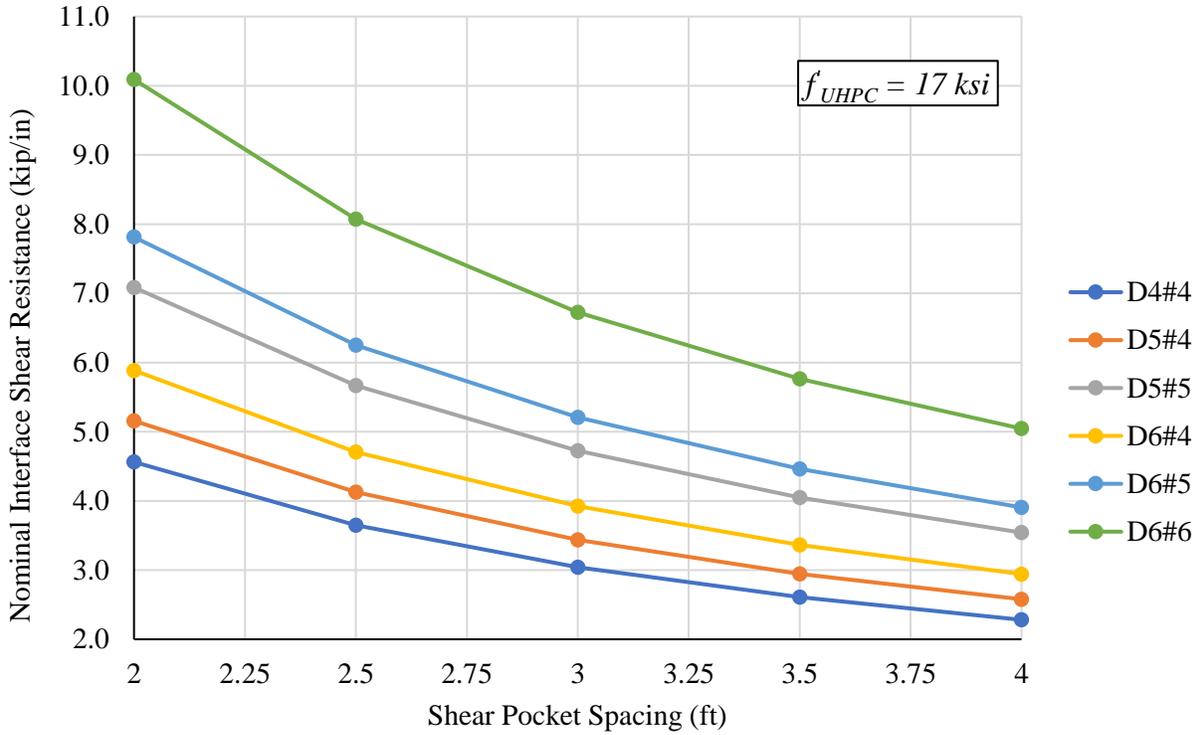


Figure 5.3: Design Chart for UHPC with Compressive Strength of 17 ksi.

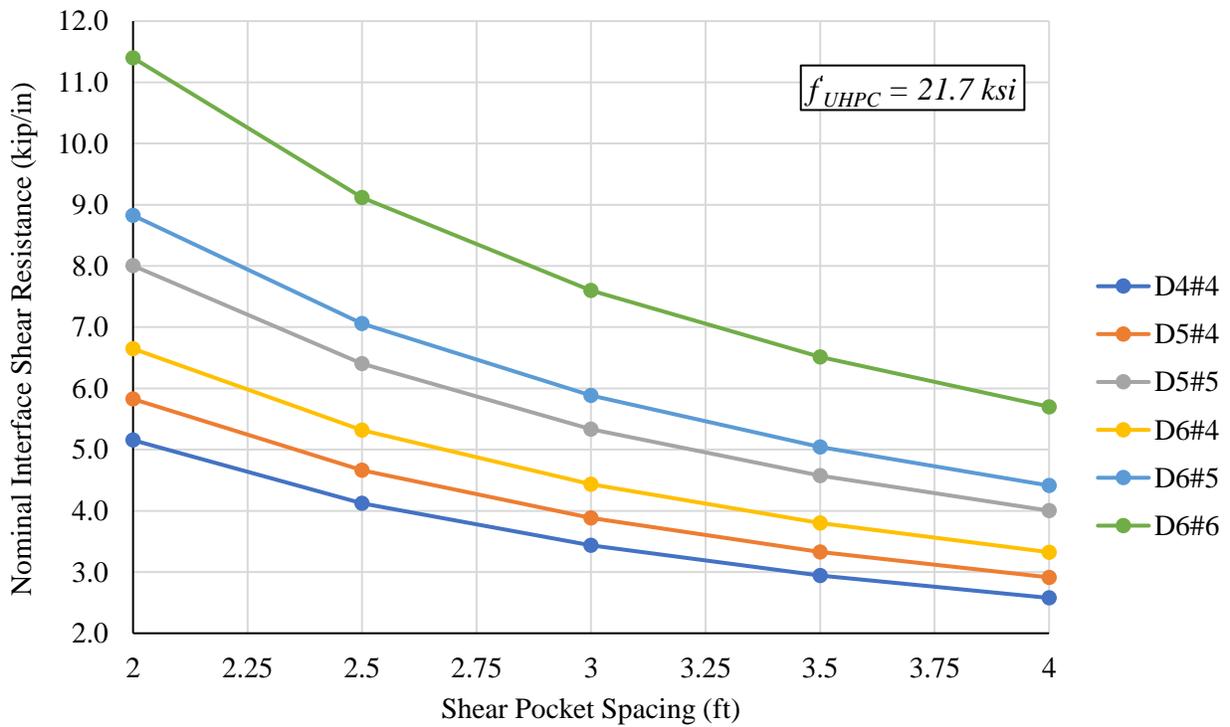


Figure 5.4: Design Chart for UHPC with Compressive Strength of 21.7 ksi.

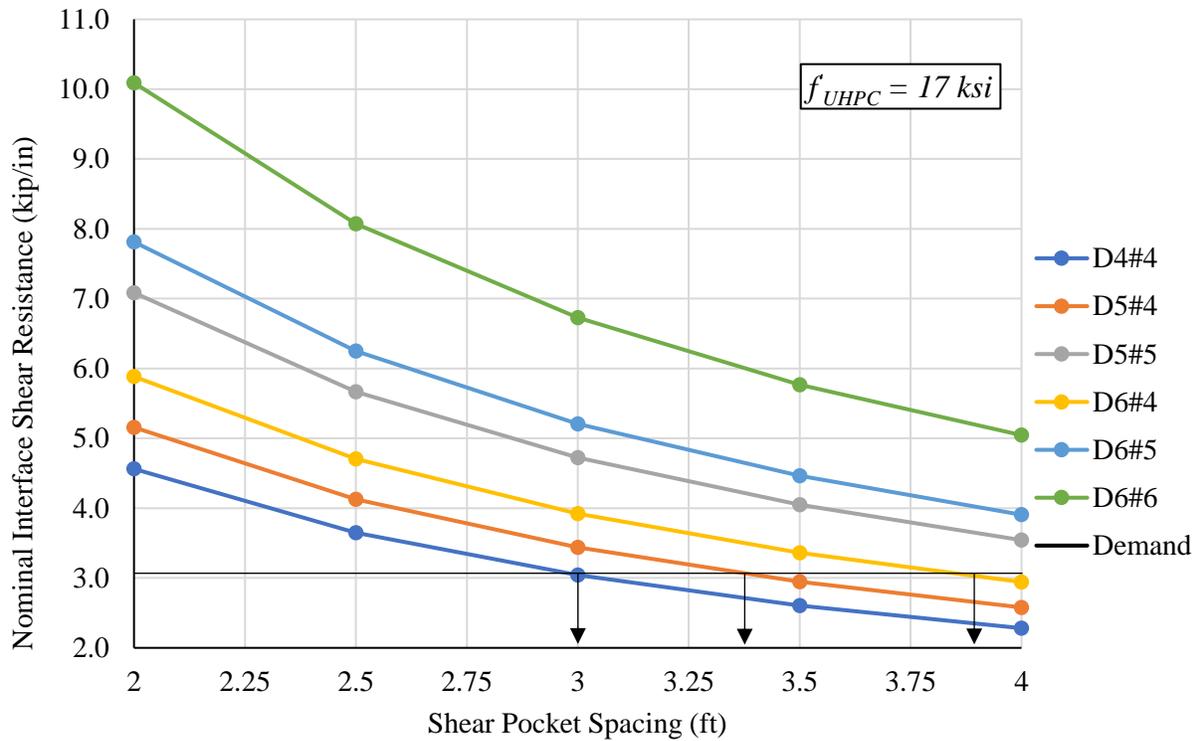


Figure 5.5: Demonstration of how to use the design aid Chart.

Table 5.01: Shear Pocket Design Alternatives for PCI BDM Ex. 9.1a.

Solution	Nominal Interface Shear Resistance (kips/in.)	f_{UHPC} (ksi)	Shear Pocket		
			Diameter (in.)	Embedded Bar Size	Spacing (ft.)
1	3.07	17	4	4	3
2			5	4	3.5
3			6	4	4

Chapter 6. Conclusions and Recommendations

6.1. Summary

Recently, the implementation of UHPC in bridge connections is growing rapidly as an innovative method of accelerated bridge construction. This report presents a new precast concrete deck-to-concrete girder connection using UHPC. This connection makes advantage of exceptional mechanical properties, flowability, and workability to eliminate any change to the precast/prestressed concrete girder (no special connectors needed). The deck panel design is simplified by using discrete round shear pockets, 4 – 8 in. in diameter, every 2 - 4 ft over each girder line. This panel design allows transverse prestressing that significantly reduces the amount of reinforcement. Also, panel transverse prestressing reduces the cracks caused by handling and transportation when using large panels. The composite action between precast deck panels and girders is achieved through filling the shear pockets and the haunch areas with UHPC. The composite action is being enhanced by adding loop bars with adequate embedment length to the shear pockets crossing the haunch area and resting on the girder. This connection design reduces the quantity of field cast UHPC compared to the existing systems.

Two critical interface shear planes control the design of the new connection. The first plane is at the girder top surface between fresh UHPC and hardened conventional concrete (CC-UHPC), which is usually an intentionally roughened surface. The second plane is at the soffit of the deck panels across the monolithic UHPC. The loop bar placed in each pocket crosses the second plane to enhance its interface shear resistance. Also, the roughened side surface of shear pocket prevents UHPC from pulling out from the shear pockets or separating from the deck panel concrete. Since current AASHTO LRFD Bridge Design Specifications 2017 does not provide equations for predicting the interface shear resistance of either monolithic UHPC or CC-UHPC, experimental investigations are conducted to understand the new connection behavior and predict the interface shear resistance of connection.

The study methodology includes two stages; experimental investigation, and design procedure. UNL UHPC mix was used as primary mix for conducting the experimental investigation and commercially available UHPC mix for some specimens. UNL UHPC mix was designed by University of Nebraska-Lincoln (UNL) research team and sponsored by Nebraska Department of Transportation (NDOT). Straight stainless-steel micro-fibers, 0.50 in. length and 0.078 in. diameter, were used in the UHPC mixes with a dosage of 2% by volume. The experimental investigation consists of small-scale testing and full-scale push-off test. Small-scale testing is conducted to evaluate the interface shear resistance at the connection critical sections; direct shear and L-shape push-off, double shear push-off testes for monolithic UHPC, and slant

shear and L-shape push-off tests for CC-UHPC. Based on the results of small-scale testing, full-scale push-off specimens simulating an actual connection are designed and tested to evaluate its structural performance and constructability. Prediction equations were obtained from the experimental investigation results and then were implemented in the design procedure for the new connection. Finally, an example bridge is presented to demonstrate connection design using the results of the experimental investigation. Design Mathcad file and design aids are provided to simplify the design of the new connection.

6.2. Conclusions

This report introduces a new precast deck-to-concrete girder connection using UHPC that eliminates any change to the precast/prestressed concrete girder and simplifies the production of deck panels. The following conclusions can be drawn based on the experimental investigations:

1. Interface shear resistance of UHPC cast on hardened conventional concrete (CC-UHPC) can be modeled using the current AASHTO LRFD shear friction model. However, different cohesion and friction coefficients should be used for different surface textures.
2. Interface shear resistance between UHPC cast on intentionally roughened conventional concrete with 1/4 in. amplitude can be predicted using a cohesion factor of 0.8 ksi and a friction factor of 1.0. The cohesion factor is much higher than that between conventional concretes 0.24 ksi, while the friction factor is the same as between conventional concretes (1.0).
3. The intentionally roughened CC interface surface with 1/4 in. depth amplitude resulted in CC failure rather than bond failure. Therefore, the compressive strength of CC is a key parameter in predicting the interface shear resistance of CC-UHPC with roughened interface surface.
4. Forming the round shear pockets using corrugated plastic pipe showed an excellent and economical solution to get a roughened surface with 1/4" depth around its surface. Other alternatives can be applied.
5. Interface shear resistance of monolithic UHPC obtained from direct shear test ranges from 4 and 8 ksi. The small size of the specimens used in this test and the possibility of having a load path other than shear could be the reason of this inconsistency.
6. Interface shear resistance of monolithic UHPC can be predicted using a cohesion and friction factors that is dependent on UHPC compressive strength as follows; $c = 0.49\sqrt{f'_{UHPC}}$ (ksi) and $\mu = 0.85\sqrt{f'_{UHPC}}$. This formula provides a cohesion and friction factors of 2.1 ksi and 3.6 for 18 ksi compressive strength of UHPC.
7. The full-scale push-off tests showed the easy of fabrication and casting for proposed connection. However, the capacity of the proposed connection is highly dependent on the stability of UHPC which can be determined by flow table test according to ASTM C230, specified by ASTM C1856. The implemented UHPC should have a flow of 8 to 10 in. to achieve the desired capacity for the connection.

8. The proposed connection is easy to fabricate, simple to erect, and economical when non-proprietary UHPC is used. It can be designed to satisfy interface shear demands in most bridges while using practical pocket size and spacing as shown in the design example.
9. A rigorous quality control (QC) procedures should be specified when using UHPC in this application as the performance of the proposed connection is highly dependent on having adequate workability and fiber stability of UHPC during casting.

6.3. Recommendations

The authors recommend the following based on the experimental investigation:

1. The shear pocket diameter is recommended to be from 4 to 8 in. and their spacing between 2 to 4 ft.
2. It is preferred to choose shear pocket design out of the different alternatives, provided by the design aid charts, that allows using the same design all over the girder to simplify the panel production and erection.
3. The shear pockets shall have a roughening surface around its surface. The corrugated plastic pipe showed an excellent and economical solution to get this roughening surface. Other alternatives can be applied.
4. The end and intermediate diaphragm shall be cast using CC to the top of the girder to fix the girders while panels erection.
5. The proper sealing of the haunch area by placing the precast panels directly on the compressible material prevents the UHPC leakage while maintain the road slope.
6. The loop bars can be either placed before or after casting UHPC. It is preferred to be placed before to ensure their implementation and prevent fiber disturbance caused by the loop bar driven in the haunch area.
7. The flowability of the field cast UHPC must not exceed 10 in. using a flow table test (10 in. diameter table) according to ASTM C230, specified by ASTM C1856, to achieve the full capacity of the connection.
8. The transverse joints should be formed with a minimum 1/2" plywood sheets higher than the top surface of the deck panel as the UHPC has a high flowability that tends to fill all the gaps inside the casting area.
9. The shear pockets should be cast starting from the highest haunch depth (lower point on the girder top) and let the shear pocket to overflow before covering it by a heavy weight or plywood.
10. The top surface of UHPC transverse joints shall be ground before the UHPC achieves a compressive strength of 10 ksi.

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