Performance of Multiple UHPC-Class Materials in Prefabricated Bridge Deck Connections

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Abstract:

Ultra-high performance concrete is gaining attention as an alternative to cementitious non-shrink grouts for closure pours in prefabricated bridge element (PBE) connections. One common method of joining PBEs is to use field-cast grout which is cast over interlaced reinforcing bars or connectors to achieve structural continuity between elements. Previous research has indicated that connections employing UHPC require shorter rebar lap splice lengths and reduced material volumes compared with those employing conventional grout materials. Furthermore, the resulting connections are more robust in regard to structural performance. This paper presents research being conducted at the FHWA Turner-Fairbank Highway Research Center on the performance of five different, commercial-available UHPC-class materials in prefabricated bridge deck connections. To evaluate the different materials, a series of large-scale precast deck panel connection tests were carried out. Each deck panel test specimen employed the same reinforcement and geometric details in the connection region. The primary difference between specimens was the type of UHPC employed in the connection. Deck panel connection assemblies were subjected to three different loading protocols, which included cyclic crack loading, fatigue loading, and monotonic loading until failure. The different loading protocols were developed to assess the bond strength between precast concrete and UHPC, the connection's resistance to cracking and damage under cyclic loading, and the ultimate load and deflection capacity of the deck panel system. Bond strength between precast concrete and UHPC was also assessed using a series of small-scale companion bond tests, which will also be discussed. Results will be presented such that comparisons can be drawn among the different UHPC-class materials. Some comparisons will also be made between the behavior of connections with UHPC and those employing conventional cementitious grouts.

Keywords: Bond strength, cracking, fatigue, bridge deck, accelerated bridge construction

1. Introduction

The performance of prefabricated bridge systems is highly dependent on the design and detailing of connections between elements. Structural continuity between elements is commonly created using field-cast grout cast over interlaced reinforcing bars. Ideally, these connection grouts are self-consolidating, have high early strength, good dimensional stability, and bond well to precast concrete. Traditionally, connections between prefabricated elements have been grouted using conventional non-shrink cementitious grouts (NSG). These grouts provide some of the aforementioned properties but may lack in others. One alternative to conventional grouts that has been gaining popularity for prefabricated bridge element connections is ultra-high performance concrete (UHPC). UHPC is emerging as a viable substitute for conventional grouts because the fresh and hardened properties of UHPC-class materials better align with the desired properties mentioned above.

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Currently, the most popular U.S. application of UHPC in prefabricated bridge construction is for connections between prefabricated bridge deck elements. Previous studies have demonstrated the structural performance of prefabricated bridge decks with UHPC connections is similar to that of conventional cast-in-place construction (Haber et al., 2016). Furthermore, the advanced properties of UHPC allows for simple reinforcement details within the connection region. Using conventional non-shrink grouts, the flexural reinforcement within the connection region typically requires hooked or U-shaped bars to meet development length requirements (Li et al., 2010). Furthermore, additional reinforcing bars are typically required to resist secondary forces such as temperature and shrinkage loads; these bars are usually referred to as “lacer” bars. Such details increase congestion within connection region and can result in poor constructability. Using UHPC, there is typically no need for hooked flexural reinforcement or lacer bars, thus greatly simplifying the detailing and increasing the constructability.

As the interest in UHPC-class materials for PBE connections grows, interested bridge owners will look to better understand how this class of materials behaves in prefabricated bridge deck connections. To meet this growing need, the structural concrete research group at the FHWA Turner-Fairbank Highway Research Center is currently conducting a study on the performance of five different, commercial-available UHPC-class materials that may be suitable for prefabricated bridge deck connections. This paper presents some of the key results from the aforementioned study. The experimental program focuses on how these materials behave in prefabricated bridge deck connections when subjected to different loading regimes. Focal areas include interface bond between UHPC and precast concrete, cracking behavior under cyclic loading, fatigue response, and monotonic ultimate loading behavior. The behavior of specimens using UHPC connections are compared amongst one another, and are also compared with a well-detailed connection using conventional non-shrink cementitious grout.

2. Experimental Program

2.1. Overview

The experimental research presented in this paper consisted of two phases: 1) interface bond behavior between precast concrete and the different UHPC-class materials; and 2) performance of the different UHPC-class materials for connection grouting between adjacent prefabricated bridge deck elements. Interface bond behavior between precast concrete and UHPC was evaluated using two different test methods and only focused on a single precast concrete surface preparation; the precast concrete surface had an exposed aggregate finish. A previous study conducted by the authors investigated the effect of different precast concrete surface preparations on the bond strength to UHPC (De la Varga et al., 2016a). Results showed that the exposed aggregate finish will maximize the bond between connection grouts and precast concrete. The performance of prefabricated bridge deck connections were evaluated using large-scale precast deck panel specimens which were subjected to three different loading protocols to assess behavior under different levels of cyclic and monotonic loading.

2.2. Materials

As previously noted, five different commercially-available UHPC-class materials were included in this study. Table 1 lists the available mix design details for each UHPC material, and also denotes the designation used to identify each UHPC in subsequent sections of this paper. It is important to note that since each product is proprietary, very little information can be provided about that type and volume of powder constituents e.g., cement, silica fume, fine aggregates. For this study, each UHPC was dosed with 2% steel fibers by volume. Table 1 provides information about the steel fibers used for each mix. Four of the five fiber types were brass coated and had tensile strengths beyond 300 ksi (2 GPa); the fibers used for material U-A were dissimilar. Each UHPC had a compressive strength between 20 and 25 ksi (138-172 MPa) after 28 days of curing at lab temperature without additional curing treatments such as exposure to stream or water bath. Additional material property data on the different UHPCs used in this study can be found in corresponding paper by De la Varga et al., 2016b.

The conventional concrete used in this study had a specified 28-day compressive strength of 6 ksi (41 MPa), was air-entrained, and used No. 57 stone course aggregate. The non-shrink cementitious grout was portland cement-based, non-metallic, and had a specified 28-day compressive strength of 8 ksi (55 MPa) when mixed for maximum fluidity. The reinforcing steel used in deck panel specimens had specified yield strength of 60 ksi (413 MPa) and met ASTM A615 requirements.

Table 1. UHPC mix details

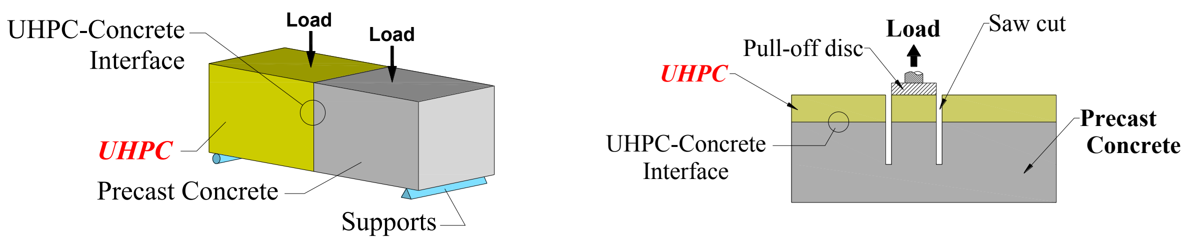
Designation	U-A		U-B		U-C		U-D		U-E	
Mix Design	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)	lb/yd^3	(kg/m^3)
Pre-blended dry powders	3503†	(2078)†	3516	(2086)	3600	(2136)	3700	(2195)	3236	(1920)
Water	278	(165)	354	(210)	268	(159)	219	(130)	379	(225)
Chemical admixtures	23	(13.7)	48	(28.7)	preblended*		89††	(53)††	73	(44)
Steel fiber content	277	(126)	88 / 179	(52 / 106)	272	(123.6)	263	(156)	156	(156)
Steel Fiber Tensile strength, ksi (MPa)	160 (1100)‡		≥305 (2100) / ≥305 (2100)		348 (2400)		399 (3750)		399 (3750)	
Length, in (mm)	1.18 (30)‡		0.5 (13) / 0.79 (20)		0.5 (13)		0.5 (13)		0.5 (13)	
Diameter, in (mm)	0.022 (0.55)‡		0.012 (0.3) / 0.012 (0.3)		0.012 (0.3)		0.008 (0.2)		0.008 (0.2)	

†: Not pre-blended but come in as separate ingredients, which include fine silica sand, finely ground quartz flour, portland cement, and amorphous micro-silica
*: The chemical admixtures were dry powders and pre-blended with other powder ingredients
††: It includes three chemicals, a modified phosphonate plasticizer, a modified polycarboxylate high-range water-reducing admixture, and a non-chloride accelerator
‡ : Fibers were straight with hooked ends and did not have a brass coating

2.3. UHPC-to-Precast Concrete Bond Tests

The bond between precast concrete and UHPC was assessed using two different methods based on current ASTM standards. Figure 1 shows an illustration of the two test methods. The flexural beam bond test (Figure 1a) was based on ASTM C78, which is originally intended for measuring the flexural tension strength of concrete using a 6 in. x 6 in. x 21 in. (152 x 152 x 534 mm) prism. This test was modified to measure the flexural tension bond strength between precast concrete and the connection grout materials. Specimens were created by first casting a precast concrete beam half, 10.5 in. (267 mm) in length, with an exposed aggregate finish on one end. The exposed aggregate finish was created using an in-form retarding on one face of the beam mold, which was pressure washed after concrete was cast and allowed to cure for 24 hours. The precast concrete half was allowed to cure for at least 28 days prior to casting UHPC against the exposed aggregate face. Prior to casting UHPC, the exposed aggregate surface was cleaned using pressurized air to remove dirt and grit, and was left dry prior to casting UHPC; thus, interface pre-wetting was not employed. Specimens were left in their molds until the predetermined test dates, which occurred once the UHPC had cured for 7- and 14-days. The test configuration and loading rates were taken directly from the ASTM C78 standard.

The second test used to evaluate bond strength was the ASTM C1583 direct tension pull-off test (Figure 1b). The test specimen consisted of a concrete base slab measuring 36 x 36 in. (914x914 mm) square by 4 in. (102 mm) deep. Similar to beam specimens, the precast concrete was allowed to cure at 28 days prior to casting a 2-in. (51-mm) thick UHPC topping upon the exposed aggregate surface of the concrete base slab. In preparation for testing, 2-in. (51-mm) diameter pull-off discs were glued to the UHPC surface, and a partial core was drilled at each disc location as shown in Figure 1b. The partial core went through the UHPC layer and 1 in. into the concrete base slab. A specialized pull-off test fixture was used to apply the load according to the ASTM C1583 standard and to record data.



(a) ASTM C78 (Modified) – Flexural Beam Test (b) ASTM C1583 - Direction Tension Pull-Off Test

Figure 1. UHPC-to-Precast Concrete Bond Test Configurations

2.4. Large-Scale Deck Panels Tests

Figure 2 shows the details of the prefabricated deck panel connection test specimens. A total of six deck panel specimens measuring 102 in. (2.6 m) in length, 28 in. (711 mm) wide, and 6 in. (152 mm) deep were tested. Specimens were reinforced with #5 (16 mm diameter) bars in the longitudinal direction and #4 (13 mm diameter) bars in the transverse direction. Furthermore, all specimens maintained the same longitudinal reinforcement ratio, and employed triangular shear keys. Five of the six specimens employed UHPC as the connection grout material, and a single specimen used conventional non-shrink cementitious grout for comparison. Figure 2a depicts the reinforcement detailing in the connection region for specimens using UHPC grout. These panels employed straight bars with a non-contact lap splice length of 5.5 in. (140 mm) and a connection width of 6 in. (152 mm). Lap splice details were designed according to the publication *Design and Construction of Field Cast UHPC Connections* (Graybeal, 2014). Figure 2b depicts the connection region reinforcement detailing for the single specimen using non-shrink grout. This specimen employed U-bars with an 8.5-in. (216-mm) non-contact lap splice length, a connection width of 10 in. (254 mm), and two transverse #4 (13 mm diameter) lacer bars.

The global specimen geometry, test set-up, and instrumentation plan is pictured in Figure 2c. Specimens were tested in a four-point bending configuration with the tension face oriented upward to facilitate visual inspection. Load was applied using a servo-controlled hydraulic actuator and a spreader beam. Each specimen was instrumented with eight LVDT displacement transducers to measure curvature at the connection interface, and a pair of strain gages that were placed on opposite reinforcing bars on the tension side of the specimen; strain gages were located within the precast deck panel approximately 1.5 in. (38 mm) from the connection interface.

Deck panel specimens were subjected to three different loading protocols, which were applied in succession. During the first protocol, referred to as "cyclic crack loading", specimens were subjected to series of cyclic load groups of increasing intensity. Load was cycled between 10% of the calculated cracking moment, $0.1M_{cr}$, and a peak load target that varied with the number of applied cycles. Five thousand cycles were applied for each upper load target which ranged from $0.3\text{-}1.2M_{cr}$. After which, an additional 50,000 cycles were applied at the $1.2M_{cr}$ peak load target. The second protocol was a fatigue loading regime where specimens were subjected to two different peak load levels. The first load level cycled 1,000 times between 10% and 73% (approximately) of the calculated yield moment M_y; these cycles are referred to "overload" cycles. The second load level cycled 99,000 times between 10% and 50% of M_y; these cycles are referred to "low-level" cycles. These two segments were repeated 20 times. Prior to applying the first set of overload cycles, specimens were subjected to 20,000 cycles of low-level loading, which served as a transition from cyclic crack loading. Specimens surviving the fatigue protocol were subsequently subjected to monotonic loading until failure.

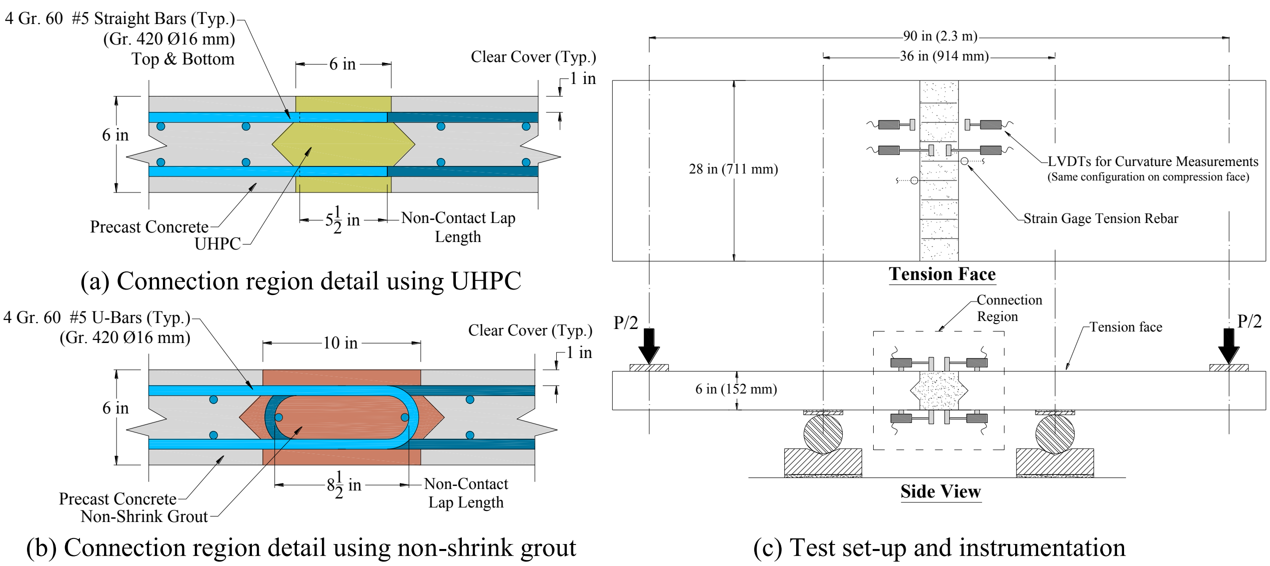


Figure 2. Specimen Details (1in = 25.4mm)

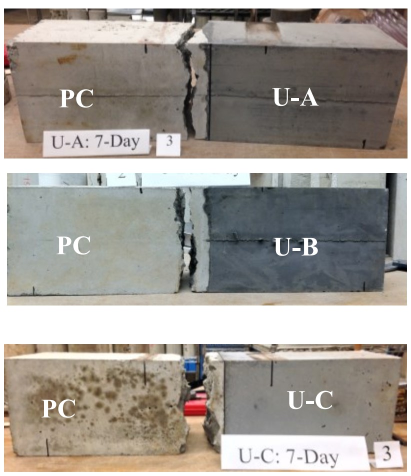
3. Results

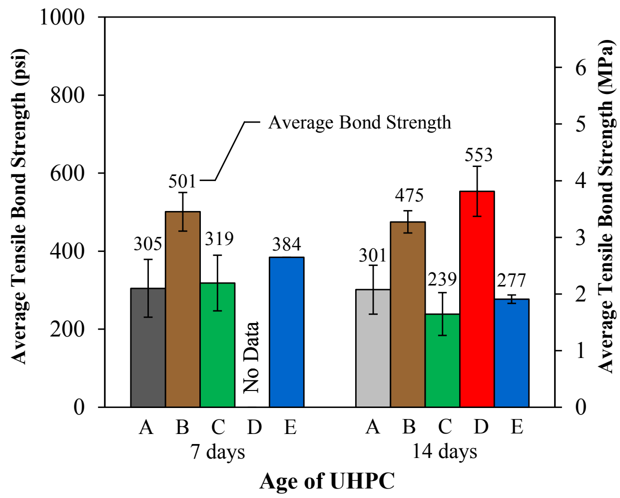
3.1. UHPC-to-Precast Concrete Bond Tests

For each age, 7- and 14-days, a set of three bond test beam specimens were tested. Eighty-three percent (25 of 30) of beam specimens failed within the precast concrete half, which indicts good bond between the two materials. Figure 3a shows a set of photos from the 7-day beam bond tests illustrating failures that occurred in the precast concrete; the precast concrete is denoted by "PC". The five specimens that did not failure in the precast concrete failed at the interface location. Inspection of these five specimens revealed that although failure occurred at the interface, there were still thin portions of precast concrete bonded to the UHPC half. This indicates that failure was not completely governed by loss of bond.

A set of five samples were tested for each age and UHPC type using the direct tension bond pull-off test. Failure of specimens was usually a result of either rupture of the substrate concrete or bond failure at the interface between the two materials. The plot presented in Figure

3b reports the average stress from samples that failed at the UHPC-concrete interface; the error bars represent +/- one standard deviation. Of the five UHPCs, products U-B and U-D had bond strengths that were similar and much higher than the remaining three products. By 14 days, both of these materials had average interface bond strengths near or greater than 500 psi (3.45 MPa). As a practical reference, the calculated tensile strength of 5,000 psi (34.5MPa) concrete is 514 psi (3.5 MPa) using the expression shown in section C5.4.2.7 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2014). The other UHPCs exhibit bond strengths below 400 psi (2.76 MPa). In general, the bond strength of these materials did not seem to increase over time. It has been suggested that bond strength is a function of concrete's maturity (Delatte et al., 2000). Thus, this result is not unexpected given that the UHPCs tested in this study did not exhibited significant compressive strength gain after 7 days of age (De la Varga, 2016). If fact, the bond strength tended to decrease slightly between 7- and 14-days. It should be noted that a set of tests were also run on the non-shrink grout material. The average 7- and 14- day bond strengths were 263 psi (1.81 MPa) and 299 psi (2.1 MPa), respectively.





(a) Photos from flexural beams tested at 7-days (b) Bond strength as measured by direct tension pull-off testing

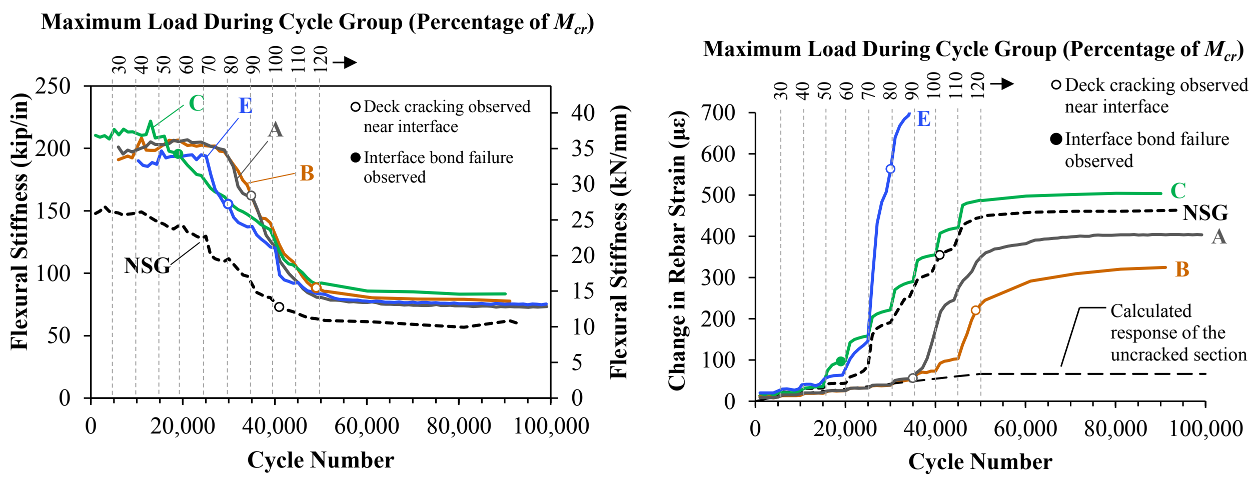
Figure 3. Result from UHPC-to-Precast Concrete Bond Tests

3.2. Deck Panels Tests

3.2.1. Cyclic Crack Loading

The primary goals of the cyclic crack loading procedure were to investigate the performance of the UHPC-to-concrete interface at the component-level scale and to investigate the cracking resistance of the connection region. Figure 4a depicts the measured degradation of flexural stiffness as a function of cycle number. The horizontal axis at the top of plot indicates the peak load target for a given set of 5,000 cycles. It can be observed that the panels employing UHPC exhibited higher initial stiffness than panel employing non-shrink grout (NSG). Lower initial stiffness in the specimen with NSG has two causes: 1) the NSG grout exhibited significant shrinkage cracking prior to mechanical loading; and 2) the elastic modulus of this material is significantly less than that of UHPC. Despite the differences in initial stiffness, all specimens exhibited a similar stiffness degradation trend. Upon completion of the cyclic crack loading protocol the specimens with UHPC were 30% stiffer than the specimen that employed non-shrink grout.

Specimens were visually inspected for cracks after each set of 5,000 cycles. Of specific interest was when the first crack occurred at the connection interface or in the vicinity of the connection interface. The results from this inspection are indicated by icons shown on the plots in Figures 4a and 4b. Only the specimen using grout U-C exhibited interface bond failure, which was observed after experiencing peak loads up to 60% of M_{cr}. The other specimens did not exhibit interface bond failure, but cracked near the interface in the precast deck panel. Although interface cracking in these specimens was not visually observed, this does not guarantee the some degree of cracking was not present. Figure 4b depicts the change in measured rebar strain near the connection interface as a function of cycle number. The change in rebar strain is defined as the difference between the maximum and minimum strain recorded for a given cycle. The plot also displays the calculated (expected) response of an elastic, uncracked section. It can be observed that the measured response for each specimen deviates significantly from the expected response prior to the observed crack marker. This could indicate cracking near the interface prior to visual observation. If this is the case, the trend shown in Figure 4b would agree with the direct tension bond strength test results. That is, the specimens with U-C and U-E grouts were the first to deviate from the expected response shown in Figure 4b, and also had the lowest interface bond strength. Whereas, specimens with U-A and U-B grouts exhibited lower rebar strains near the interface and had higher measured bond strengths.



(a) Degradation of flexural stiffness

(b) Measured rebar strains near the connection interface

Figure 4. Results from Cyclic Crack Loading

3.2.1. Fatigue Loading

The primary goal of the fatigue loading protocol was to induce damage without causing fatigue fracture of the steel reinforcing bars. Figure 5 shows the relationship between flexural stiffness degradation and number of load cycles. The data presented in this plot only reflects measurements taken during the low-level fatigue cycles. As noted previously, a set of 20,000 low-level cycles was applied prior to the first set of overload cycles. During these first 20,000 cycles, there was very little stiffness degradation. A sizable drop in stiffness can be observed immediately after the 20,000 cycle mark, which is a result of the first set of overload cycles. The abrupt loss of stiffness is a result of newly formed cracks and propagation of existing cracks. In specimens employing UHPC connections, new cracks formed within the precast concrete deck sections. The specimen with non-shrink grout (NSG) also exhibited new cracks, but these cracks occurred both within the connection and within the precast concrete sections. Comparatively speaking, all specimens exhibited very little stiffness loss in the cycles following the first overload set. Figure 6 shows photos of specimens NSG and U-C after completion of fatigue loading. The apparent damage for specimen U-C is representative of that observed for specimen U-B. Each specimen exhibited similar, uniformly-distributed crack patterns within the precast concrete deck panel sections. Visual inspection of the UHPC connection grouts revealed no damage apparent to the naked eye. A crack microscope was used to further inspect the UHPC connection grouts and revealed that a few fine microcracks were present with crack widths less than 0.001 in. (0.025 mm). The specimen employing non-shrink grout exhibited a significant amount of cracking within the connection grout, which can be observed in Figure 6. The cracks within this region could be observed by the naked eye, and had crack widths greater than 0.01 in. (0.25 mm).

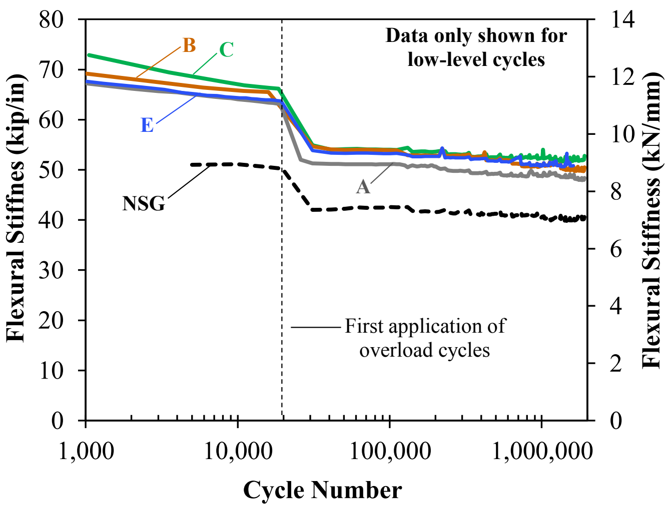


Figure 5. Degradation of Flexural Stiffness During Fatigue Loading

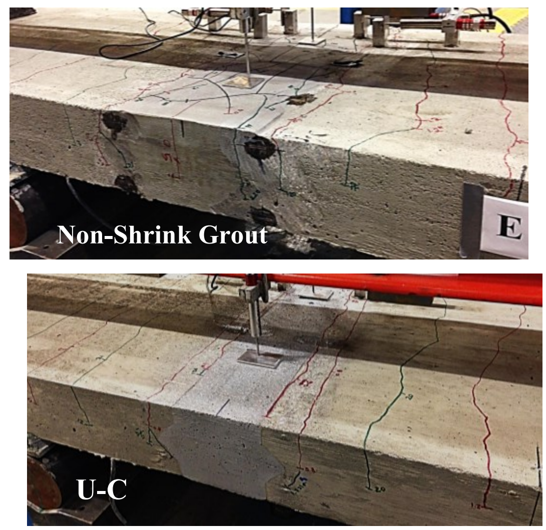
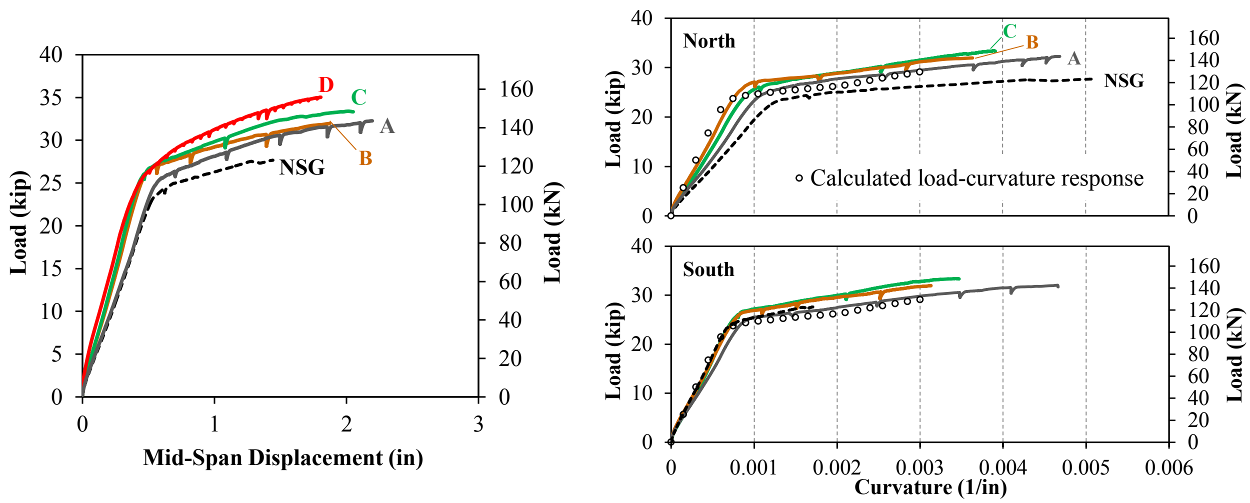


Figure 6. Cracking in the Connection Region after Cracking and Fatigue Cycles

<u>3.2.1. Ultimate Loading</u>

The ultimate loading force-displacement relationships are shown in Figure 7a. Each curve was truncated at the point of peak load for comparison purposes. With the exception of the specimen with grout U-A, specimens employing UHPC connections exhibit approximately the same initial stiffness, apparent yield point, and have similar ultimate strength and ultimate displacement. The approximate displacement ductilities (ultimate displacement / apparent yield displacement) for specimens U-A, U-B, U-C, and U-D were 3.8, 3.5, 3.8, and 3.4, respectively. The specimen with non-shrink grout exhibited a ductile force-displacement response, but differed slightly from the responses exhibited by specimens with UHPC. The specimen with non-shrink grout had lower initial stiffness, ultimate strength, and ultimate displacement. The approximate displacement ductility of this specimen was 2.3, which was significantly lower than that of the specimens employing UHPC connections grouts. Specimens employing UHPC grout failed as a result of precast concrete crushing, and the specimen with non-shrink grout failed as a result of non-contact lap splice failure. This is further illustrated in Figure 7b which shows the load-curvature relationships measured over the two interface locations; north and south correspond to the left-and right-hand interface locations depicted in Figure 2c. For reference, a set of markers indicate the calculated (expected) response using a simple moment-curvature analysis. At both interface locations, the specimens with UHPC connections behave similar to one another and show good agreement to the expected response. For the specimen with non-shrink grout this is not the case at the north interface. At this location, an excess amount of curvature is measured for a given load level, which would correspond to slippage of the reinforcing bars.



(a) Force-displacement relationships (b) Load-curvature measurements at the connection interface

Figure 7. Ultimate Loading Response of Deck Panel Specimens (1 in. = 25.4 mm)

4. Conclusions

This study focused on the behavior of different commercially-available UHPC-class materials for prefabricated bridge deck connections. The bond behavior between precast concrete and different UHPC-class materials was evaluated using two different methods, and performance of the different UHPC-class materials was evaluated in a series of large-scale deck panel connection tests. Results were compared with the performance of a conventional non-shrink grout system.

It can be concluded that flexural beam and direct tension pull-off bond tests exhibited different results for the same materials and test periods. Flexural tests indicated that UHPCs bonded to an exposed aggregate precast concrete surface has enough bond strength to produce failure within the precast concrete. Whereas the pull-off bond test indicates lower bond strengths, it directly assesses tensile bond and may be a more conservative approach to evaluating bond strength between UHPC and concrete. Based on this method, UHPCs may have bond strengths as high as 500 psi (3.4 MPa) and as low as 200 psi (1.4 MPa). Furthermore, results from direct tension pull-off tests were confirmed with results from cyclic crack loading, which indicated that some UHPC-class materials may be more prone to interface bond failure than others.

In general, prefabricated bridge deck connections using UHPC will have higher initial stiffness and better stiffness retention compared with connections using conventional non-shrink grout materials. Furthermore, UHPC connections have good resistance to fatigue and exhibit minimal damage even at high cycle counts. Post-fatigue ultimate loading indicated that UHPC connections exhibit ductile failure modes and good displacement ductility. In summary, besides bond behavior to precast concrete, the different UHPC materials investigated in this study behaved very similarly in these component-level tests.

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