Fabrication and Experimental Verification of a UHPC Decked I-Beam in Ontario, Canada

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Primary Topic Area: Components and Structures
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Abstract

As a follow up to the paper on the design of the UHPC decked I beam (DIB) presented at the Second International Interactive UHPC Symposium, the current paper will look at the DIB fabrication and testing. Fabrication and testing were completed as part of Prestressed/Precast Concrete Institute's (PCI) study on the implementation of UHPC in precast long span structural elements in buildings and bridges. Due to lack of UHPC structural design codes, there is a need to perform experimental structural evaluation of these precast structural elements, including small scale specimens and full scale 49 ft. [15 m] beam. The experimental findings and comparisons with design codes will be presented.

Keywords: Ultra high performance concrete; bridges, precast, testing, code, fabrication

1.0 Introduction

Precast prestressed concrete I-beam are the most dominant beam shapes in the US and Canada. They offer optimization of materials used, and high flexibility in framing arrangements. Once the forms are available, production is relatively simple, and handling and erection are generally efficient, even for spans as long as 60 m (200 ft). Staged construction and future bridge widening is also easier to accommodate with the I-beam than with other girder shapes. When considering ultra-high performance concrete (UHPC) with its relatively high unit cost of materials, it becomes even more important to take advantage of this common bridge girder shape. In addition, the durability of UHPC and a recent move towards accelerated bridge construction (ABC), are factors that make combining an I-beam and deck into a single cross section highly favorable. Decked I-beams have been popular in the northwest region of the US, including Alaska, due to their ABC benefits. Combining the concept of decked I-beam with the concept of a waffle (ribbed) deck slab system has resulted in the new UHPC-NUDIB cross section shape. The shape was initially introduced in Ontario for the contractor FACCA, and was incorporated into the comprehensive PCI-UHPC Project what was started in 2018 and concluded in 2021. The shape is highly optimized in terms of minimizing the volume of UHPC used. The top flange skin is designed to resist truck wheel loads plus an additional wearing layer. The total depth of the top flange is comparable to that of conventional decks, and thus, the stiffness and deflections are comparable. The total depth of the UHPC-NUDIB is also comparable to that of a conventional concrete superstructure. Thus, stiffness and strength are maintained and durability is superior.

A significant disadvantage of the UHPC-NUDIB is the relative difficulty in forming. However, creating steel forms is a one-time process, and is amortized over hundreds of uses and many years. In this paper, it is shown how the forms were created, and how a test specimen was successfully produced and shipped from Ontario to South Carolina for full scale testing.

Design with UHPC is still in its early development stages. The PCI-UHPC Project gave a set of recommendations, which were used to design the bridge for which this product was created, and to predict theoretical behavior of the full-scale specimen under various loading conditions.

Results of the experiments confirmed as conservative, yet simple, the guidelines offered for design with UHPC. Results include shear, flexure, punching shear, transverse direction behavior, and end zone behavior, with global shear and flexure the focus of this paper. The experience and experiments documented in this paper clearly show that one can successfully build with this highly optimized UHPC bridge superstructure shape while achieving all required design criteria, including first-cost economy and durability.

2.0 Design Considerations

Flexural behavior may be considered as one of the clearest positive characteristics of UHPC. Ultimate limit states should be checked for all flexural members to satisfy required capacities. Additionally, service limit states should be checked to assure no cracking under service loads, as service conditions often control the design of prestressed members. Vertical shear is perhaps the least understood characteristic of UHPC, but a critically important parameter in terms of structural optimization. The combination of high compressive strength and significant tensile strength in UHPC equates to a high shear strength that can allow for elimination of conventional shear reinforcement.

A lack of UHPC structural design codes and a need to optimize the beam section, such as a minimizing the web width and utilizing a waffle slab deck with thin surface skin, there is a need to perform experimental evaluation of the newly introduced precast structural element.

Small-scale specimens were developed and tested; however, they are not the focus of this paper, which will present only the full-scale beam testing results. A summary of design considerations informed by small tests are as follows:

1. Web bending: The lateral bending of the 4 in. [102 mm] thick beam web may be a critical aspect during shipping, erection, and the casting of the joint, especially if heavy equipment is used on the deck before the longitudinal joint has gained adequate strength. After the joint has set up and the deck has gained continuity, web bending web is not anticipated to be critical.
2. Punching shear: It is essential for the 3 in. [80 mm] waffle deck skin to be able the transfer wheel loads to the ribs of the waffle deck.
3. Positive & negative moment in the deck top flange: Once the design truck wheel loads are transferred to the ribs of the waffle deck, the deck is assumed to act like a series of T-beams transferring the deck loads to the girder webs.

Full-scale tests were undertaken to investigate the global flexure, global shear, and global deck behaviors. A full-scale beam having the dimensions of the UHPC decked I-beam (DIB) presented at the second international interactive UHPC symposium was used for testing. That is, a decked I-beam of approximately 49 feet [15 m] long with an 8 ft-8 in. [2.6 m] wide top flange, a 3 ft-3 in. [991 mm] deep, with a 4 in. [102 mm] thick web. The deck is ribbed in the transverse direction with a 3 in. [76 mm] skin and a total flange thicknesses of 9 in [229 mm].

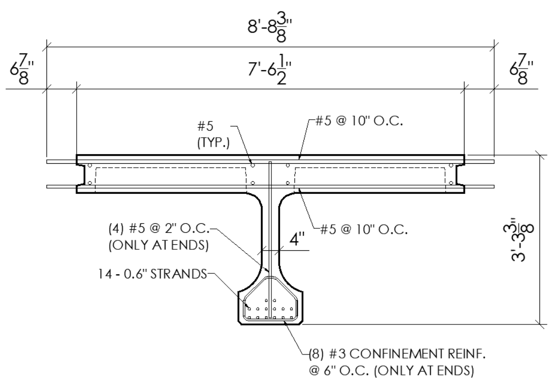

Figure 1: Full-Scale UHPC DIB Tested, cross-section (left), 3D printed model shows waffle slab (right)
Note: 1 ft. (') = 305 mm and 1 in. (") = 25 mm

The test specimen was produced with a total of fourteen (14) 0.6-in. diameter 270 ksi prestressing strands [15 mm]. Each end of the specimen was reinforced with four (4) vertical #5 [16 mm] bars at 2-in. spacing [51 mm] and eight (8) #3 [10 mm] closed confinement bars at 6-in. [152 mm] spacing. Six (6) strands were anchored into a UHPC abutment at each end of the DIB.

Additionally, for one-half of the beam, shear reinforcement equal to #3 bars [10 mm] at 10-in. [254 mm] spacing is placed to model the minimum required shear reinforcement for what is believe to be the longest practical span, 121 feet [37 m], this depth of member is believed to be capable of. The other half of the beam had no shear reinforcement other than the four (4) vertical #5 [16 mm] bars provided for bursting reinforcement.

3.0 Dura® UHPC Mix Design

The flow and consistency properties of the produced mixtures were tested as recommended by the Canadian Standards Association (CSA) A23.1 Annex U (2019), ASTM C1437 (2020), and ASTM C1856 (2017) standards. The average flow diameter of the produced mixture ranged from 180 mm to 220 mm (7 in to 8.7 in). Curing and weather protection started shortly after the pour process was completed and extended up to seven days. After the required compressive strength at transfer was achieved (minimum 80 MPa [11.6 ksi]), the strands were cut and the DIB was demolded. Then, the DIB was moved to an ambient controlled facility until subsequent steam curing. Within 10 days of casting, the DIB was exposed to steam curing for at least 48 hours at a temperature range of 85 to 95ºC [185 – 203 ºF].

A UHPC with a minimum 28-day steam-cured strength of 150 MPa [21.8 ksi] was designed and implemented as the precast material. Mixture proportions with respect to cement mass are listed in Table 1. The steel fiber content was 2 percent by volume.

Table 1: Mixture Design and Proportioning with Respect to Cement Mass

Material Trade Name	Cement	Supplementary Cementitious Materials	Sand	Water	High Range Water Reducer	Workability Modifier
Dura® UHPC	1	0.23	0.67	0.23	0.04	0.007

4.0 Fabrication

The DIB fabrication process consisted of preparing, mixing and placing Dura® UHPC, provided by Dura Concrete Canada Inc., into special forms which include steel, plastic, and styrofoam. Figure 1 showed a 3D rendering of the DIB which can be related to the formwork photographs in Figure 2. The forms were cleaned, oiled and prepared, including running and stressing the strands. The formwork was heated to a minimum of 10 ºC [50 ºF]. Temperature sensors were attached to the reinforcements at four (4) different locations.

Figure 2: Photographs of Full-Scale DIB Formwork

For mixing the Dura® UHPC, a high shear Skako mixer was used. Ice and workability admixture were used to slow down the hydration process and to maintain the flow and workability. The mixing sequence includes adding powdered materials to the mixer. After performing dry mixing of the preblended premix for about 2 minutes, water and chemicals were introduced. When a plastic mixture was achieved, steel fibers were added. The produced UHPC was discharged to traditional concrete buckets and was moved to the precast bed facility. A special chute was built to convey the UHPC from the bucket to the girder bottom flange and web. External vibrating equipment on the outside of the web formwork was used as required to aid the UHPC flow.

Figure 3: Chute for Casting (left) and Transport Bucket Loading Chute (right)

When the girder top flange/deck were cast, the surface was levelled using a spike roller. To create a sutiable deck finish, a fine ribbed rubber matting was placed on the surface after levelling. Then, a heavy steel roller was moved back and forth on top of the rubber matting to force the matting ribs into the concrete. The matting ribs are oriented parallel to the traffic direction, which enables vehicular traction on an exposed bridge deck.

Figure 4: Smoothing the Concrete Surface (left) and Placing Ribbed Matting (right)

In general, the fabrication process satisfied the Canadian Prestressed Concrete Institute (CPCI) and Canadian Standards Association (CSA) requirements in terms of dimensional tolerances, prestress and post-stress loading procedures, UHPC mixing, fabrication sequence, curing regimes, and temperature monitoring.

Figure 5: UHPC DIB Ready for Shipment from Ontario, Canada to North Carolina, USA

5.0 Experimental Verification

The full-scale DIB produced in Ontario, Canada was shipped to North Carolina State University in Raleigh, NC, USA for experimental testing. The beam was tested in a variety of ways. First, in flexure to the service, factored, and above-factored levels to examine the bending behavior up to cracking. Second, in shear to failure at each end to examine the global shear capacity of the end with internal steel stirrups and of the end without any internal concentrated steel reinforcement. Third, under concentrated punching loads to simulate wheel loads on the thinnest portion of the deck skin. And forth, under concentrated loads at the flange cantilever tips to test the transverse bending capacity of the flange itself. All tests were successful with applied loads reaching levels expected in the design. Tests on global flexure behavior and global shear behavior will be the focus of this paper.

5.1. Testing for Global Flexure Behavior

The primary purpose of testing in flexure was to observe the behavior of the DIB at an approximate factored-level bending moment. In addition, behavior at service-level moments were observed on the way to the factored level. It was also decided during testing to push the applied loads beyond the factored-level moment (by about 20%), in an attempt to slightly crack the specimen in flexure. Cracks were not observed visually, even at this high level of applied load, but the recorded data indicated a distinct change in stiffness likely correlating with flexural cracking. Importantly, the flexure tests stopped and unloaded prior to substantially damaging the beam, so as to preserve the integrity of the structure for subsequent shear testing (at each end) and local testing of the deck.

For flexure testing, the specimen was simply supported at its ends on 1” [25 mm] thick by 12” [300 mm] long steel bearing plates supported by 2” [50 mm] diameter steel rollers. The rollers in turn rested on steel plates and supporting beams which rested on the laboratory floor (Figure 6).

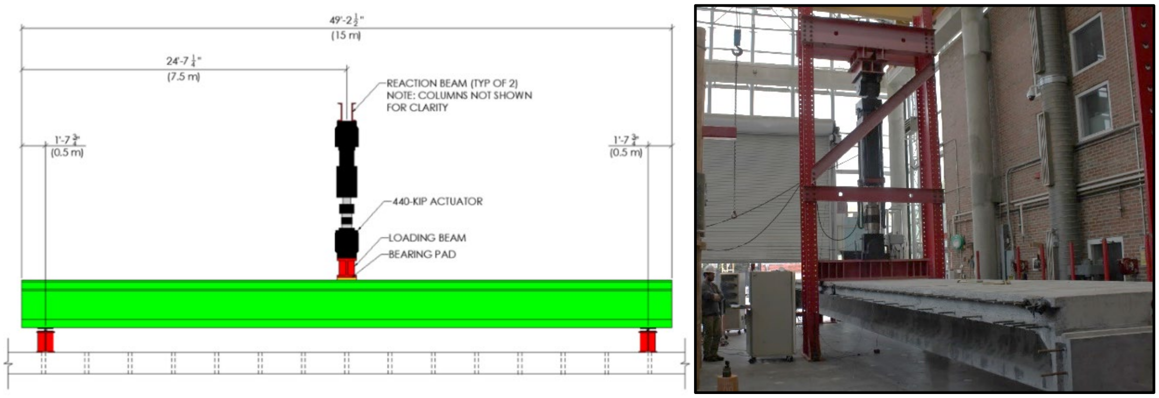

Figure 6: UHPC DIB Flexure Test Schematic (left) and Setup in the Laboratory (right)

The load was applied at mid-span using a hydraulic actuator supported by a braced reaction frame. Load was transferred to the DIB using a stiff steel beam spanning the transverse direction across the top flange. A 1” [25 mm] thick plywood bearing pad was placed between the loading beam and the DIB to ensure even load distribution between the loading beam and the UHPC deck. Applied load, vertical deflection, and longitudinal strain were measured.

The specimen was loaded and unloaded incrementally to the selected moment levels outlined in the charts below. The applied load was held at each peak level for the short period of time necessary to examine the specimen and to document/photograph any observations (typically less than 5 minutes at each load point).

The specimen exhibited near-perfect elastic behavior in the loading/unloading stages up to the factored load (left side of Figure 7). Near the factored load, some non-linearity was observed, however, there was no indication of cracking at this stage. At the load level of 1917 k-ft [2599 kN-m] and beyond (right side of Figure 7), audible popping and cracking noises were noted during loading, however, no visible cracks were located. A general stiffness degradation in the load-deflection behavior indicates that cracking likely occurred. After unloading from the flexure test, the beam recovered to nearly completely (about 1 mm residual) and was visually undamaged. Importantly, the beam remained in sound condition for shear testing at the end of the flexure test.

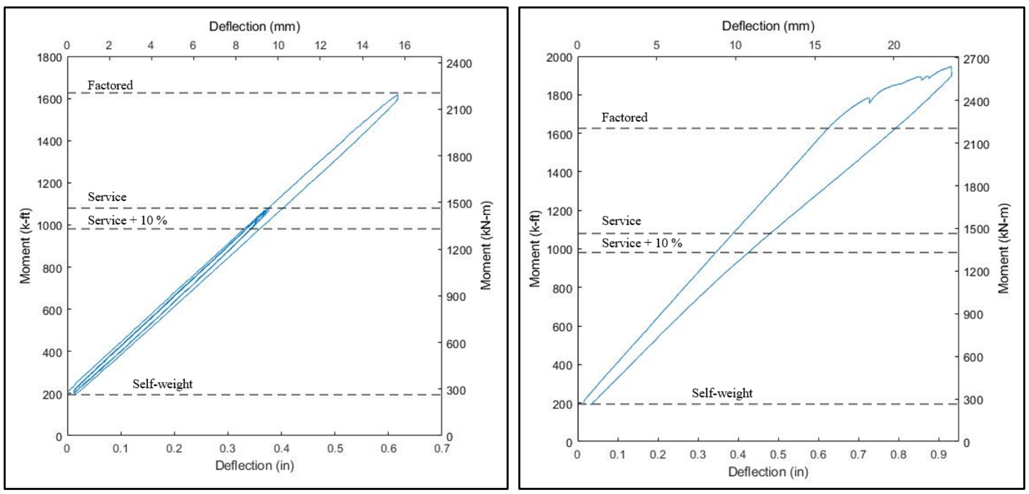

Figure 7: Moment vs. Deflection Plots for to Factored Load (left) and the Cycle beyond Factored (right)

5.2. Testing for Global Shear Behavior

The test setup for flexure was used again for shear by simply sliding the applied load towards the end of the beam. For each shear test, the applied load was centered 8'-3" [2.5 m] from the end of the beam to create a span-depth ratio of 2.5. After testing one end of the beam to failure, the applied load was moved to the opposite end for a second test to failure. In this way, the end of the beam with internal steel stirrups was compared to the end of the beam without internal steel stirrups. Both tests loaded and unloaded a beam end incrementally to selected load levels, including service, factored, cracking, and theoretical ultimate capacity. Loading continued in each test to failure.

The end without stirrups was tested first, with a factored design load was 156 kips [695 kN] and theoretical ultimate capacity of 259 kips [1150 kN]. The end exhibited near-perfect elastic behavior past the factored load, up to approximately 209 kips [929 kN], where the first signs of diagonal shear cracking were observed with well-distributed hairline cracks (~0.005"; 0.1 mm). Crack angles were mostly between 25 and 30 degrees. Distributed cracking progressed with loading, until the primary failure crack formed at around 360 kips [1600 kN]. This crack was also quite flat in angle and spanned nearly the entire distance from the support to the centerline of the load. The crack continued to widen until the specimen failed at a peak applied load of 414 kips [1840 kN], well in excess of the capacity predicted by the design method. Close inspection of the failure crack revealed that the steel fibers provided significant crack bridging.

The end of the beam without stirrups was tested next with the same factored load, but a predicted ultimate capacity of 297 kips [1319 kN]. The end also exhibited near-perfect elastic behavior in loading/unloading steps up to the calculated cracking load (209 kips; 929 kN) where few hairline diagonal shear cracks were observed. Compared to the test without stirrups, cracking was less at equivalent load steps. Crack angles mostly ranged from 35 to 40 degrees. However, flexural cracking was observed to initiate in the end region at an applied load of 225 kips [1000 kN]. The flexural cracks corresponded with stirrup locations, and continued to propagate as loading continued, until most (if not all) of the 14 prestressing strands ruptured in the end region at a peak applied load of 437 kips [1940 kN]. Strand rupture began to develop at a load of 430 kips [1910 kN] and a deflection under the load point of 3.9 inches (99 mm).

Figure 8: Shear Tests without Stirrups (left) and with Stirrups (right)

6.0 Conclusions and Lessons Learned

In conclusion, this paper demonstrates that the UHPC DIB design can be successfully cast, handled, and shipped using conventional equipment. Testing confirmed excellent structural performance of the optimized section with test values far exceeding design values and predictions. As design codes are improved, and confidence with UHPC increased over time, it may be possible to further optimize this member. Shear capacities exceeded 2 times predicted failure loads, even without stirrups, and shear behavior without stirrups was more traditional. Thus, the use of UHPC without discrete steel reinforcement is promising and should be considered as a design option where appropriate.

7.0 Acknowledgement

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