ULTRA-HIGH-PERFORMANCE CONCRETE IN STANDARD PRECAST/PRESTRESSED CONCRETE PRODUCTS

Marc Maguire, Dept. of Civil Engineering, University of Nebraska—Lincoln
George Morcous, PhD, Dept. of Construction Systems, University of Nebraska—Lincoln
Kromel Hanna, PhD, Dept. of Civil Engineering, University of Nebraska—Lincoln
Maher K. Tadros PhD, Dept. of Civil Engineering, University of Nebraska—Lincoln

ABSTRACT

Ultra-High Performance Concrete (UHPC) is a new class of concrete that has superior performance characteristics compared to conventional concrete. Recent research projects have focused on developing bridge girders with unique shapes and dimensions to efficiently utilize UHPC. These girders require special and relatively complex forms that lead to a substantial increase in the production cost in addition to the already high material cost of commercial UHPC products.

The paper presents the outcomes of a research project sponsored by the Precast/Prestressed Institute Daniel P. Jenny Fellowship to investigate the use of UHPC in standard precast/prestressed products. In this project, UHPC double tee girders are proposed for bridge superstructures because of their structural efficiency and economy of production. In addition, the large 0.7 in. diameter strands and Grade 80 welded wire reinforcement are used to enhance the flexural and shear capacities of the proposed section while eliminating costly steel fibers. In order to evaluate the structural performance of the proposed system, two full-scale test specimens were fabricated at Coreslab Structures (Omaha), Inc. and tested to determine their flexural and shear capacities. The specimens were 51 ft long, 23.75 in. deep, made of an economical non-proprietary UHPC, and reinforced with 20-0.7 in. diameter strands. Testing results have indicated the superior structural performance of the proposed design. The paper presents, detailing, fabrication, and testing of the UHPC bridge girders.

Keywords: Ultra-High-Performance Concrete, Double Tee Bridge Girders, 0.7 in. Diameter Strands, Welded Wire Reinforcement.

INTRODUCTION

Reactive powder concrete, later known as Ultra-High Performance Concrete (UHPC), developed in France approximately 12 years ago, is a new class of concrete that has superior performance characteristics compared to conventional concrete. The enhanced strength and durability properties of UHPC are mainly due to optimized particle gradation, which produces a very tightly packed mix, use of steel fibers, and extremely low water to powder ratio. The Association Francaise de Genie Civil (AFGC) in its *Interim Recommendations for Ultra-High Performance Fibre-Reinforced Concrete* and the Japan Society of Civil Engineers (JSCE) in its draft *Recommendations for Design and Construction of Ultra-High Strength Fiber Reinforced Concrete Structures* define the UHPC as a cementitious composite that has a compressive strength in excess of 21.7 ksi (150 MPa), and containing steel fibers for ductile behavior[1,2].

Recently, several proprietary UHPC products have become commercially available, such as BSI by Eiffage, Cemtec by LCPC, and Ductal by Lafarge, which is the only commercial product available in North America. On the other hand, several research efforts have focused on non-proprietary UHPC mixes that have comparable performance but are made more economical by the introduction of ultrafine particles other than silica fume as well as large aggregate. For example, the UHPC mixes developed in Ma and Schneider in 2002 with 28-day compressive strengths up to 21.9 ksi[3]. Quartz flour and silica fume were used as supplementary cementitious materials in these mixes to make the gradation curve more continuous and improve its flowing ability. Important observations of this UHPC include higher autogenous shrinkage and a different relationship between compressive strength and modulus of elasticity when compared to conventional high performance concrete, both due to the high paste content. Similarly, another research team investigated the replacement of the more expensive cement and silica fume with fly ash and ground granulated blast furnace slag[5]. It was found that the incremental addition of different combinations of fly ash and slag yielded a gradual decline in compressive strength. However, drops of compression strengths greater than 10% were not noted until a replacement of 40% for blast furnace slag and over 20% for fly ash[4]. Another related study incrementally replaced the characteristic fine sand and small parts of cementitious material with a local natural aggregate with a maximum size of 5/16 in. Little change was noted from the replacement on either compressive or flexural strength as for a constant water to cementitious material ratio .

Due to the unique strength and durability properties of UHPC, extensive work is being done to efficiently utilize this new material in precast concrete products. This work has led to the development of an optimized bridge section known as the MIT Pi-girder[6,7,8]. The Pi-girder shown in Fig. 1 requires much less concrete volume than the conventional double tee bridge girder. The equivalent solid slab thickness is 6.25” compared to 10” for the conventional double tee. The Pi-girder has so far been proposed to be made with the proprietary UHPC mix named Ductal and marketed by Lafarge Inc. The product has been primarily tested by the FHWA in McLean, Virginia[9] and Iowa DOT[10]. Ductal is shipped to precasters in three separate components: preblended dry materials, steel fibers, and chemical admixtures. The mix requires special mixing and curing procedures to achieve the expected 20 to 30 ksi strength and other performance characteristics. The cost of these components is approximately $1,000/yd^3, which is over 10 times the cost of conventional concrete mixes. In addition, the unique shape and dimensions of the Pi-girder section lead to a substantial increase in the production cost due to the need for special and complex forms. Another disadvantage is that United States plants are not set up for 48 hours of intense heat curing before the removal of the product from the prestressing bed. This alone could double the cost of a product.

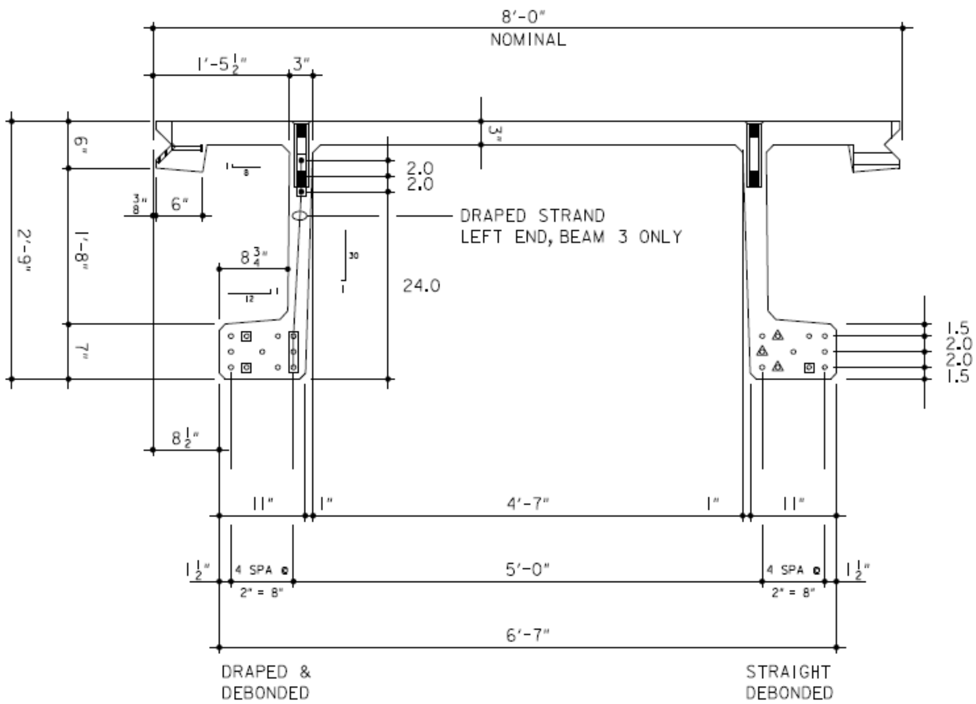


Fig. 1 First Generation of Pi-Girder[10]

Many states, such as Texas, Washington, Nebraska and New England, currently have their own double tee shaped girders for use in short to medium span bridges. Adopting one of this state exclusive girders may create resistance from both owners and precasters. Therefore, it is proposed in this project to use one of the PCI standard heavy section bridge double tees because of its wide use and availability.

The *general objective* of this research is to promote the use of UHPC in standard precast/prestressed concrete products. The s*pecific objectives* are to develop a UHPC mix that is optimized in terms of both the material and production costs. While the mix may not have the same level of compressive strength, it should be superior to those currently available and much less expensive than proprietary mixes. Also, to investigate the application of the developed UHPC mix, in combination with 0.7 in. diameter strands and welded wire reinforcement (WWR), to a standard concrete product that is readily available to precast producers.

UHPC MIX DEVELOPMENT AND TESTING

Several attempts to develop non-proprietary UHPC mixes were made at the University of Nebraska-Lincoln (UNL) since 2006[11,12,13]. However, the adopted definition of UHPC is different from that of AFGC and JSCE. The so-called NU UHPC is defined as the concrete mix that has a minimum release strength of 12 ksi, specified 28-day compressive strength of 15 ksi, self-consolidating, material cost of less than $250 per cubic yard, and performance characteristics superior to those of the High Performance Concrete (HPC) mixes currently used in Nebraska.

Table 1 shows the two mixes that were selected based on their mechanical properties, and material and production cost. Table 1 also shows their material cost that was calculated using typical prices in Nebraska as of 2008, which were $90/ton for Portland cement, $600/ton for silica fume, $15/ton for class C fly ash, $10/ton for fine sand, $15/ton for limestone, and $15/gallon for the high-range water-reducer (HRWR).

Table 1 Selected NU UHPC Mixes and their Material Cost

Component	NU UHPC Mix #4		NU UHPC Mix #5	
	Quantity (lb/cy)	Cost ($/cy)	Quantity (lb/cy)	Cost ($/cy)
Fine Sand	2075	10	1580	8
1/4" Limestone	-	-	672	5
Cement Type III	1120	50	1050	47
Class C Fly Ash	240	2	300	2
Silica Fume	240	72	150	45
HRWR	68	115	44	75
Total Water	242	-	242	-
TOTAL		$250.00		$182.00

Fig. 2 plots the results of compressive strength testing versus age and the curves that best fit the data points[14]. The figure clearly indicates the consistency of test results and the steady gain of compressive strength with time. Test results also indicate that the average compressive strength of each mix exceeded 12 ksi at 24 hours and 15 ksi at 28 days. The testing was performed on end ground, 4 in. x 8 in., cylinders with 24 hours of heat curing followed by moist curing until the time of testing.

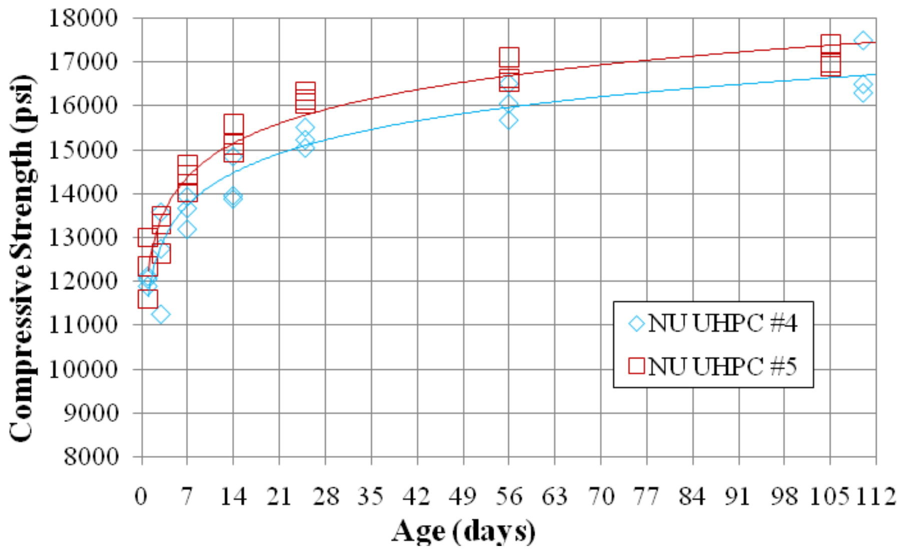


Fig. 2 Compressive Strength versus Age for Selected NU UHPC Mixes

Fig. 3 shows the results of modulus of elasticity (MOE) testing at 28 days[15] and the values calculated using the American Concrete Institute (ACI) 318-08 equation of section 8.5.1. The calculated MOE values are based on the average compressive strength at 28 days and unit weights[16] of 148 lbs/ft^3 and 149 lbs/ft^3 for mixes #4 and #5 respectively. Fig. 3 indicates that the calculated MOE is approximately 19% higher than the measured values, which is in agreement with the findings of other research programs on HPC and UHPC concretes[3,17].

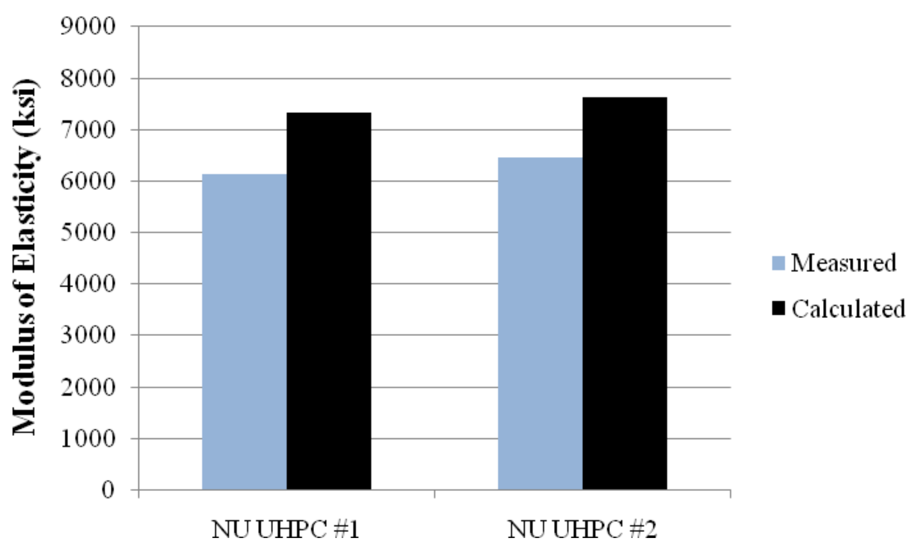


Fig. 3 Modulus of Elasticity of Selected NU UHPC Mixes

Fig. 4 shows the results of split tensile strength testing[18] and those calculated using the ACI 318-08 equation from section 8.6.1. Splitting stress calculations were based on the average compressive strengths at 28 days for mixes #4 and #5. Fig. 4 indicates that the calculated value is very close to the measured value for mix #5, while it is 10% higher than the measured value for mix #4. This indicates that the splitting tensile strength of UHPC can be adequately predicted using the current code equation.

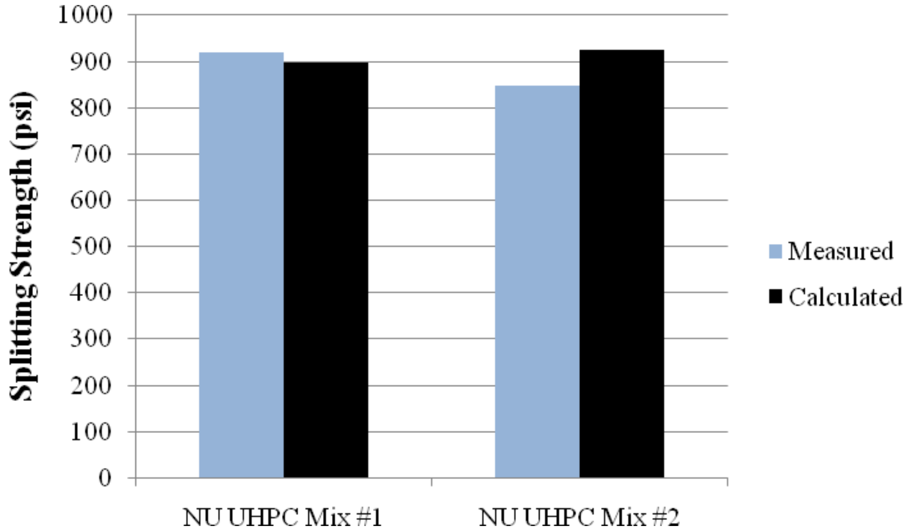


Fig. 4 Splitting Tensile Strength of Selected NU UHPC Mixes

Fig. 5 shows the results of flexure testing at 28 days[19] and the values of the modulus of rupture (MOR) calculated using ACI 318-08 equation from section 18.3.3 for Class U and Class T flexure members. Calculations for MOR were based on the average compressive strengths at 28 days for mixes #4 and #5. Fig. 5 indicates that the measured MOR of mix #5 is within the calculated range, while the measured MOR of mix #4 is approximately 10% higher than the upper limit of the calculated range. This indicates that the MOR of UHPC can be adequately predicted using the current code equations, which is consistent with previous HPC research[17].

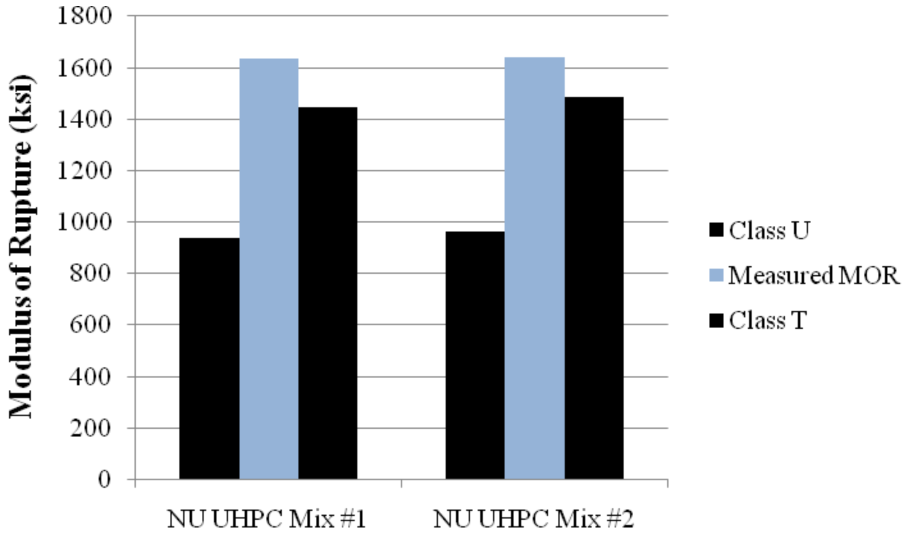


Fig. 5 Flexure strength of selected NU UHPC mixes

The National Cooperative Highway Research Program (NCHRP) Report 496 presents a method for measuring the shrinkage of concrete for prestress loss calculations[20]. This method was used to measure the shrinkage of the two selected NU UHPC mixes since their primary purpose was the production of prestressed concrete girders. Four concrete specimens, measuring 4 in. x 4 in. x 24 in., were prepared from each mix using steel molds. Five detachable mechanical (DEMEC) strain gauge disks were attached to the two opposing sides along the specimens' length at approximately 3.94 in. spacing with the center DEMEC disk centered along the length. This allowed for six readings per specimen (readings are taken every other point) using a DEMEC dial gauge caliper. The specimens were then cured at room temperature with a relative ambient humidity of approximately 70%. Readings were taken each day during the first week, once a week during the first month, once per month for three more months and a final reading at 8 months. Fig. 6 plots the measured shrinkage strains versus time for the two mixes.

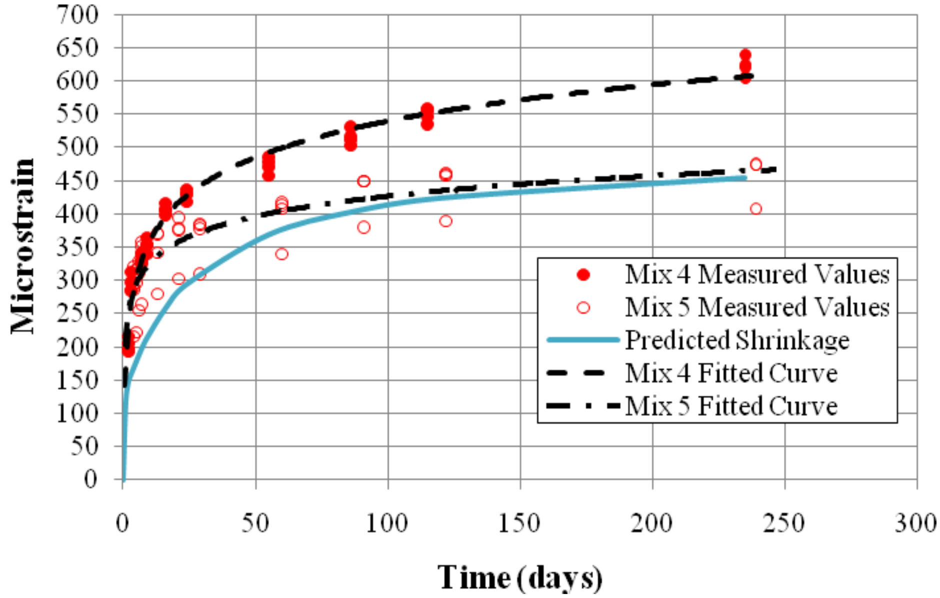


Fig. 6 Shrinkage Strain versus Time for NU UHPC Mixes

Fig 6 also shows the shrinkage strain calculated using the method proposed in the NCHRP report 496, which was adopted in the 2007 AASHTO Load Resistance Factor Design (LRFD) Specifications. Comparing the measured shrinkage strains against the predicted strains indicates that the current method provides a reasonable estimate of the long-term shrinkage strain for NU UHPC mix #5, while it significantly underestimates the shrinkage strain for NU UHPC mix #4. This is mainly due to the absence of coarse aggregate and high content of cementitious materials in mix #5. Also, it should be noted that the current method was developed for concrete strengths up to 15 ksi, while the strength of the developed mixes exceeds this limit. However, this finding is in agreement with other research performed on UHPC, where shrinkage strains were found to be on the same order[3,5].

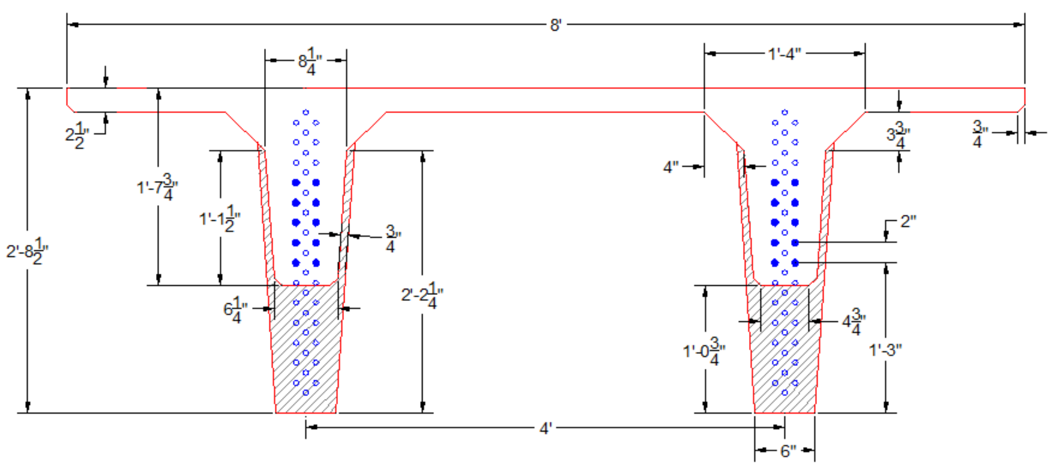
TESTING SCOPE AND SETUP

In order to develop efficient, economical, and practical precast/prestressed concrete girders for short and medium span bridges, standard PCI bridge double tees were selected. This is because of their ease of production and availability to many precast producers. The self stressing capabilities of double tee beds and ease of stripping make them ideal for fast paced and economical fabrication. Test specimens were fabricated by Coreslab Structures (Omaha), Inc., NE.

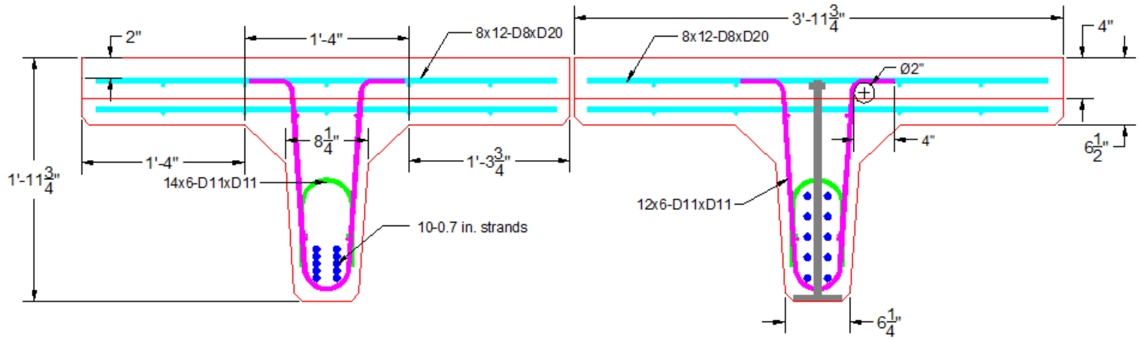
The testing program consists of six load tests performed on two full-scale prestressed concrete single tee bridge girders cast in the double tee form. The main objectives of the testing program are: 1) investigate the flexural and shear capacities of NU UHPC bridge double tee girder; 2) evaluate the shear transfer between the NU UHPC girder and the cast-in-place (CIP) topping; and 3) verify the transfer and development length of 0.7 in. diameter strands in NU UHPC.

GIRDER DESIGN AND FABRICATION

A 51 ft long simple span bridge girder was designed according to the 2007 AASHTO LRFD specifications. The precast double tee was 19.75 in. and the cast-in-place structurally composite deck was 4 in., for a total depth of 23.75 in. Fig. 7(a) shows the cross section of the double tee form used in production. Wooden blockouts were used to achieve the required depth and thickness of the stems (see hatched area in Fig. 7(a)). Because the girder is longitudinally reinforced with 20 Grade 270 low-relaxation 0.7 in. prestressing strands, 20 holes were enlarged from 0.5 in. diameter to 0.7 in. diameter to accommodate the large size (see blue dots in Fig. 7(a)). The strands were tensioned to only $0.60f_{pu}$ and located as shown in Fig. 7(a) to accommodate the bed capacity and its centroid elevation. Strands were then depressed at 0.4L as shown in Fig. 7(b). This figure also shows the detailing of the double tee cross section at the end section (left stem) and mid-span section (right stem). The girder was reinforced using Grade 80 welded wire reinforcement (WWR). Vertical shear reinforcement and confinement reinforcement were D11@6 in. along the entire length with D8 cross wires for anchorage. End zone reinforcement was 3–¾ in. headed coil rods welded to the base plate as seen in Fig. 7(b).



(a)



(b)

Fig. 7 a) Cross section of the form; b) Cross section of Girder, Left Stem: Mid Section, Right Stem: End Section

The girders were fabricated using NU UHPC #5 because of its lower cost and superior performance. A special mixing procedure was followed to ensure adequate properties of the fresh concrete. In this procedure, all dry materials (i.e. aggregates, cement, fly ash and silica fume) were mixed for 2-3 minutes. Then water and all HRWR were added simultaneously after adjustment of water quantity for the water content of the aggregates. After mixing for 10-12 minutes, a slump flow test was performed and additional HRWR was to be added if an inadequate spread (less than 25 in.) was exhibited. Following this procedure and a slightly modified mix had resulted in self-consolidating concrete with an average spread of 30 in.

Fig. 9 plots the compressive strength versus age of the precast concrete girders. The design strength of 15 ksi at 28 days was significantly exceeded in both girders. However, girders were not released until the fourth day because of the low temperature and inadequate curing, which resulted in a slow gain of strength.

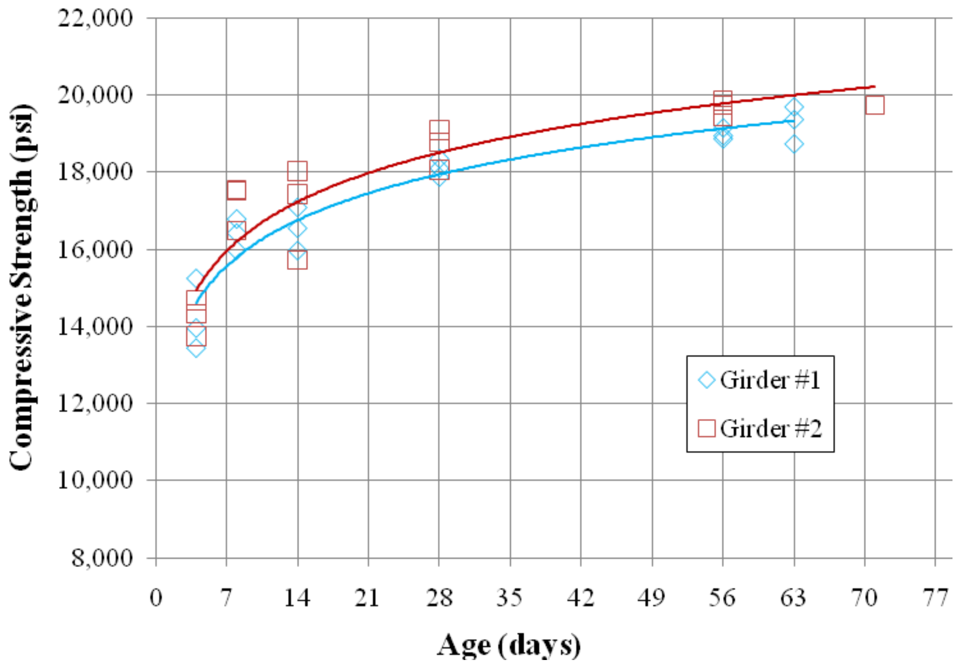


Fig. 8 Compressive Strength versus Age Plot for both Girders

The MOE is reported for both 28 days and at the day of testing. Split tensile, MOR and Poisson's Ratio values were found at 28 days and for the NU UHPC only, and can be found in Table 2 along with MOE values at each reading for both the deck and NU UHPC.

Table 2 Measured Concrete Mechanical Properties

	Girder 1	Girder 2	Deck
28 Day Compressive Strength (ksi)	17.36	17.45	-
Final Compressive Strength (ksi)	19.71	19.74	7.81
28 Day MOE (ksi)	6,677	6,520	-
Final MOE (ksi)	6,960	6,980	5,146
28 Day Poisson Ratio	0.224	0.222	-
28 Day MOR (psi)	1,167	1,088	-
28 Day Splitting Strength (psi)	1,008	997	-

GIRDER INSTRUMENTATION AND TEST SETUP

Because of the difficulty to place Detachable Mechanical (DEMEC) gauges on the sides of double tee stems before release, DEMEC gauges were placed along the top flanges of the girders, near the center line of the stems. Each girder end had 16 DEMEC gauges placed approximately 3.94 in. apart for a total of 14 reading per end. Strain readings were taken immediately prior to release, 30 minutes after release, and after 14 days.

The girder was instrumented for full scale load testing, as can be seen in Fig. 8. The deflection was measured using string potentiometers (ST-POTs) located directly under the loading points. Spring potentiometers (SP-POT's) were used to measure the end slippage of the strands nearest the loading. Electrical resistance strain gauges (ERSGs) were used to monitor the difference in strain between the cast-in-place (CIP) and the precast (PC) girder.

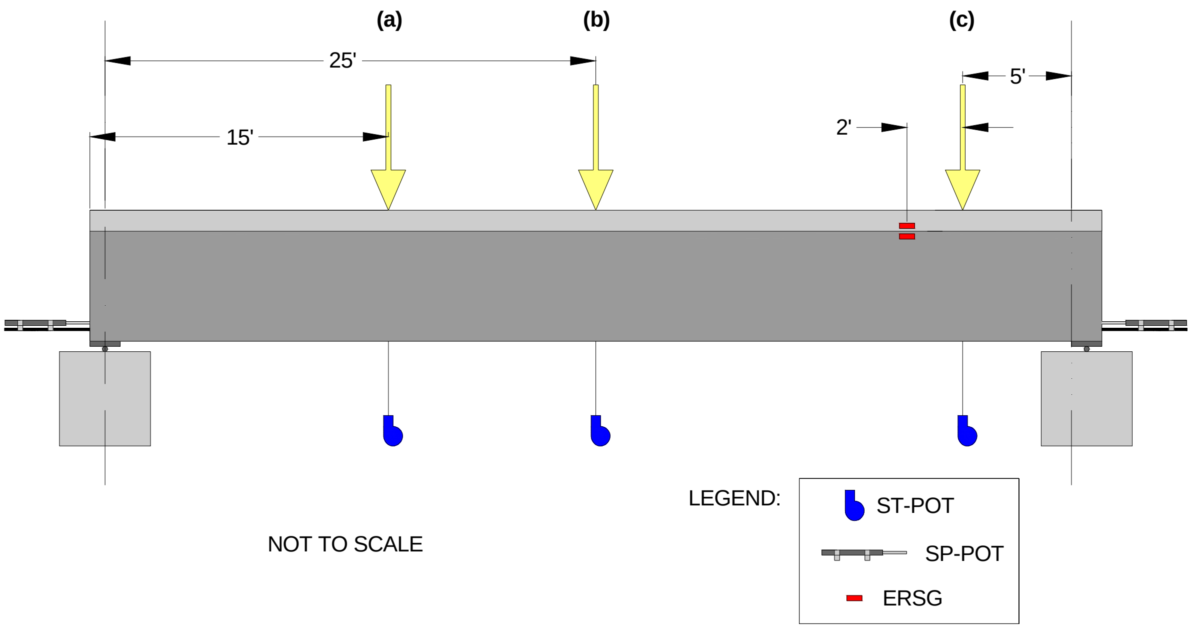


Fig. 9 Overview of Test Instrumentation for (a) Development Test, (b) Midspan Test, (c) Shear Test

For both the development length test and the midspan test, the deflection was measured directly under the load using a ST-POT, and the bottom row of strands were instrumented with SP-POTs to identify any strand slippage.

For the first shear test, a ST-POT was used to monitor the deflection directly under the point load and the slippage was again measured using SP-POTs. However, for the second shear test, a concrete diaphragm was cast around the end of the girder with the strands bent vertically, preventing any monitoring of the strands. This was done to prevent a bond failure, similar to many state's standard practice. In addition to the potentiometers, 2 ft from the centerline of the load, towards midspan, ESRGs were used to monitor the longitudinal strains above and below the CIP to PC interface. Each of the ESRGs were oriented horizontally and located 0.25 in. vertically from the interface and 0.5 in. away from each other.

TEST RESULTS AND DISCUSSION

TRANSFER LENGTH

Transfer length values were recorded at release and 14 days following release. It has been well documented that the transfer length typically expands 10% to 20% over time[23], with the majority of the extension coming in the first 14 days[24]. Therefore, these two values were considered the initial and final transfer lengths for the girders.

Results from the DEMEC strain readings were plotted versus their position along the girder along with a line indicating the 95% Average Maximum Strain (AMS) line and a best fit line of the ascending/descending branch of the strain plot. A typical strain plot can be found in Fig. 10 below. The transfer length values determined from each of the four plots, with accompanying ACI and AASHTO predictions, are tabulated in Table 3. The transfer lengths were calculated with a modified 95% AMS method[24] where the constant strain region of the plot is visually identified and reduced to 95%. The ascending/descending branch is also visually identified and a best fit linear curve is applied. The intersection of the 95% AMS line and the best fit curve is then calculated using the general slope intercept equation.

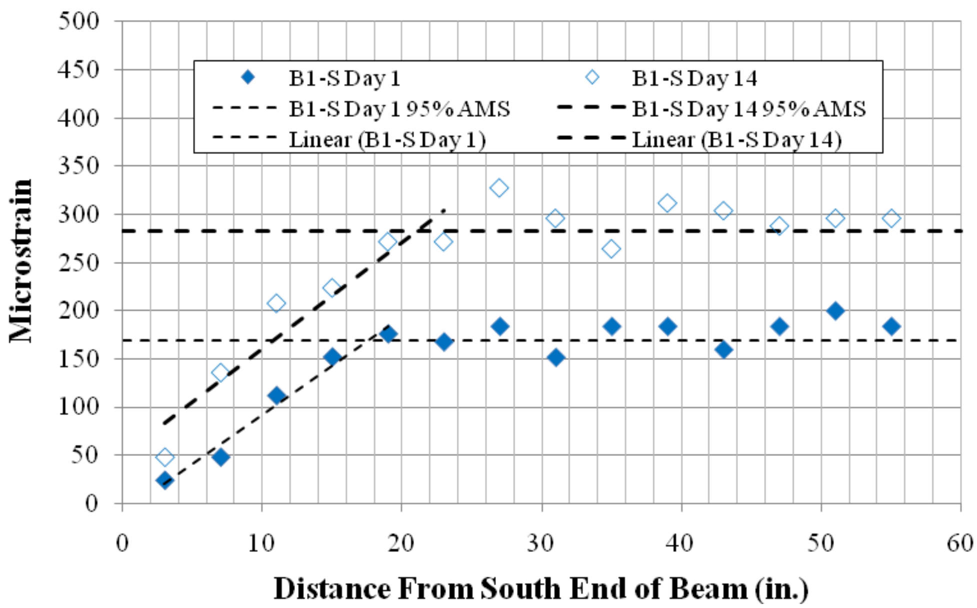


Fig. 10 Typical Transfer Length Strain Profile – Girder 1, South End

Table 3 Transfer Length Comparison

Girder End	Initial Measurement (in.)	Final Measurement (in.)	$(f_{se}/3)d_b$ (in.)	ACI, $50d_b$ (in.)	AASHTO, $60d_b$ (in.)
B1-S	17.5	21.1	33.1	35.0	42.0
B1-N	20.4	18.2			
B2-S	14.5	17.6			
B2-N	13.6	16.9			
Average	**16.5**	**18.5**			

As can be seen in Table 3, the measured transfer lengths are significantly below both the ACI and AASHTO predicted values. This was expected by the researchers, as both research experience and literature indicate exceptional bond properties of UHPC[25,26].

DEVELOPMENT LENGTH TESTING

The load deflection curves for girders 1 and 2 are shown in Fig. 11. It was predetermined that a bond failure, which would indicate a development length less than the loading point, would result in a slippage reading of 0.01 in. on the lower level of strands. The girder was not loaded to failure, due to the planned three test regimen for each girder. However, the girder was loaded to its ultimate predicted load using measured material properties of 2338 kip-ft, where the bottom strands were calculated to reach 262 ksi. There was no sign of slippage from the SP-POTs up to the calculated load. This indicates that the AASHTO prescribed development length of 15 ft is conservative for 0.7 in. strands in NU UHPC tensioned to $0.6f_{pu}$.

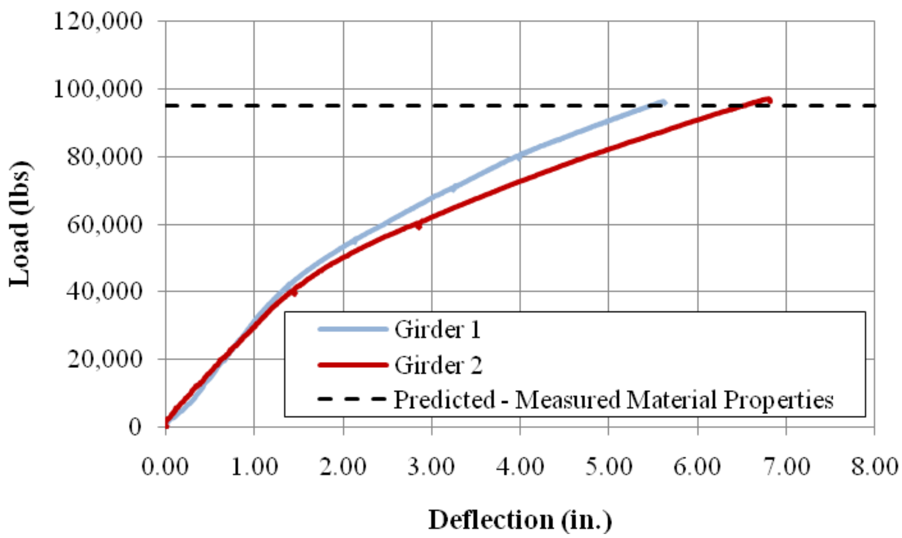


Fig. 11 Load versus Deflection Plot for Development Length Tests

ULTIMATE FLEXURE TESTING

The load versus deflection plot of the midspan ultimate flexure tests is shown in Fig. 12 and includes both girders. No significant slippage was noted throughout the test, even though the girders had been subjected to the development length tests. Both girders failed at approximately 91 kips. Using strength design and the material properties obtained from testing, it was predicted that the girder's capacity was 85 kips using the strength design/strain compatibility.

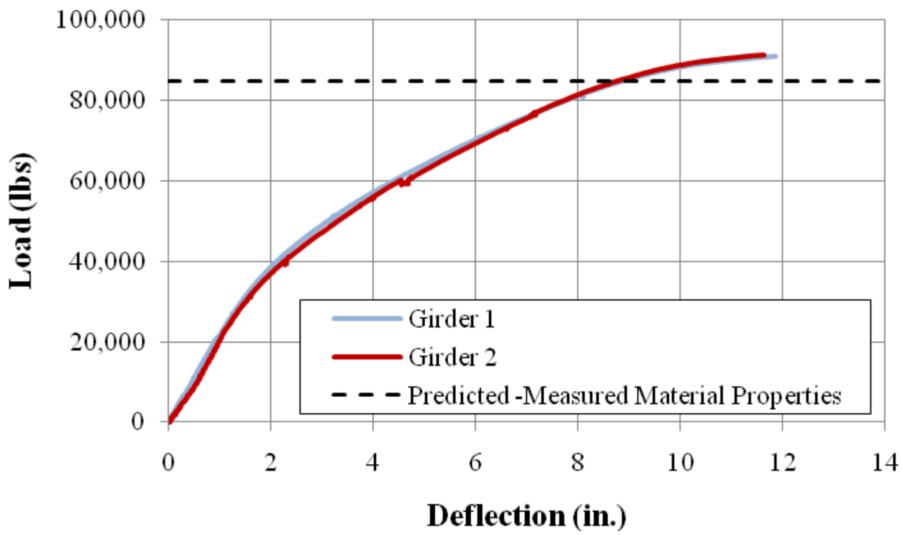


Fig. 12 Load versus Deflection Plot for Midspan Tests

SHEAR TESTING

The load versus deflection for the shear tests was plotted in Fig. 13, and it can be seen that both tests exhibit very similar behavior for the majority of the tests. The difference between the two shear tests was the addition of a CIP diaphragm surrounding the extended strands to prevent strand slippage. A sudden failure was observed for both specimens; however distinctly different behavior was observed at the end of the plot in Fig. 13 and in the failure modes of the girders. It is important to point out that both girders safely held 180 kips plus their dead load, which is equivalent to 2.5 times the total weight of the design truck.

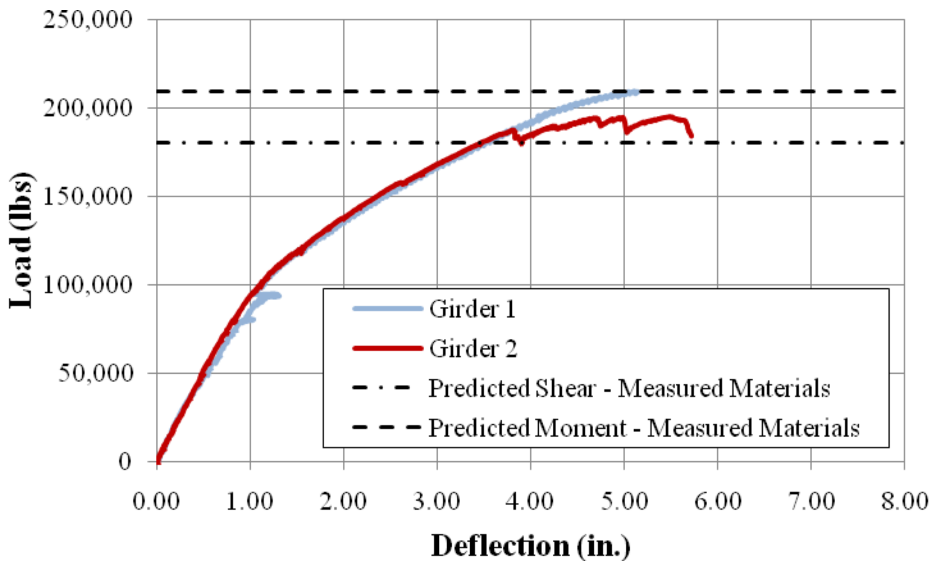


Fig. 13 Load versus Deflection Plot for Shear Tests

Girder 1 exhibited a classic bond failure, which agrees with the bottom strand average slippage values, plotted in Fig. 14. It can be observed in Fig. 14 that the bottom strands started slipping at around 185 kips. Both gauges slipped well beyond the predetermined limit of 0.01 in. for a bond failure. It can be said conclusively that the cause of failure for this test was the bond of the strands to the concrete. For this reason, it was decided that for Girder 2, the protruding strands were to be bent upward and cast into a diaphragm similar to many state's standard practice for bridge girders, which was expected to force a shear failure.

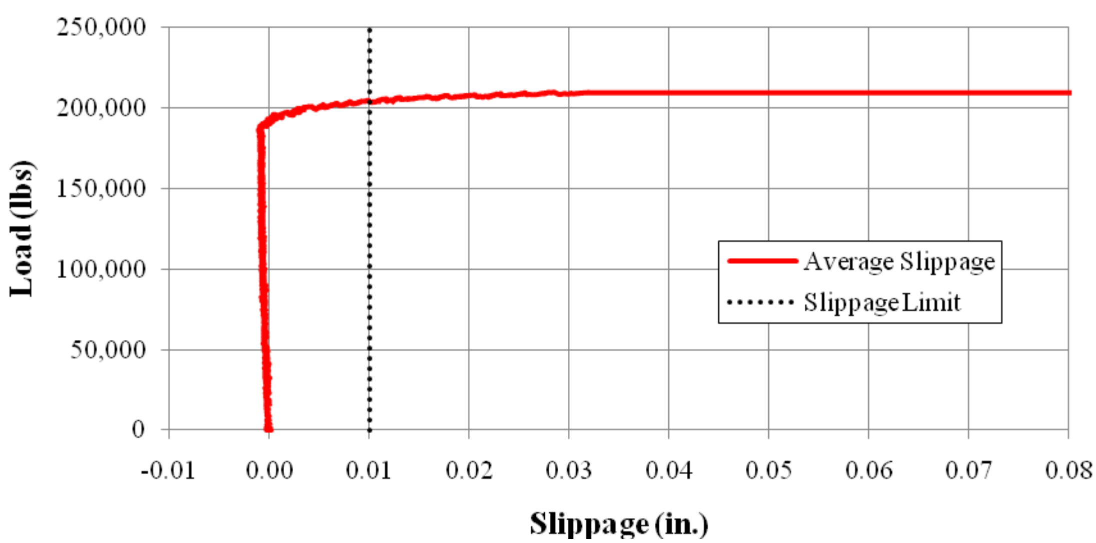


Fig. 14 Load versus Slippage Plot for Girder 1 Shear Test

The sudden failure of Girder 2 can be attributed to the delamination and total separation of the topping from the precast section. This was both observed during the test and through the measurement of the longitudinal strain using the ERSGs placed on either side of the CIP/PC interface, which can be found in Fig. 15.

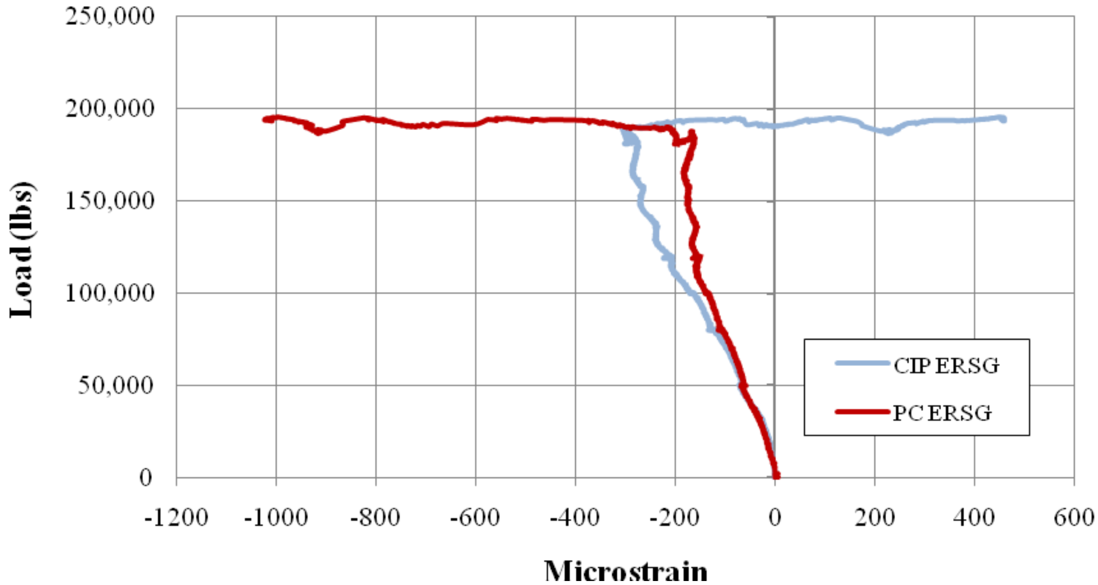


Fig. 15 Load versus Longitudinal Strain on either side of CIP to PC Interface

Predictably, the strain in both gauges began compressive (negative) with the CIP gauge higher than the precast gauge. However, near 180-185 kips the PC gauge gained a lot of compression very quickly until failure at 191 kips. Conversely, the CIP gauge, in the same load range, lost all compression and shifted to tension before failure. This sudden divergence of the strains, measured at only 0.5 in. apart vertically, indicated composite action up to approximately 185 kips and a sudden shift to non-composite afterwards and up to failure. This explains the poor performance relative to Girder 1. From these observations it was determined that the shear failure observed for Girder 2 was initiated by a horizontal shear failure. Fig. 16 displays both the horizontal and vertical shear failure exhibited by Girder 2.

Fig. 16 Shear Failure of a) Girder 1, and b) Girder 2

This type of horizontal shear failure was not unexpected by the researchers, as nearly every test performed on the girder was accompanied by some sort of minor delaminating of the girder from the deck. This is due to the very smooth surface of the NU UHPC. The interface could not be intentionally roughened to create a good bond due to the unique early age properties of NU UHPC. The surface was sand blasted similar to many building products in an attempt to increase the bond. In this instance of very high horizontal shear stresses, created by 2.5 times the design truck weight, surface preparation may not have been enough.

CONCLUSIONS

The purpose of this research was to investigate the application of UHPC in standard precast/prestressed concrete products. A non-proprietary UHPC mix was proposed to be used in the production of full-scale bridge girders. The mix consisted primarily of locally available materials and eliminated the use of random steel fibers, which is a major cost item in proprietary UHPC mixes. Grade 80 welded wire reinforcement (WWR) was used instead for shear reinforcement. The developed mix was attainable using practical and affordable batching, mixing, and curing procedures and costs less than $250 per cubic yard. Material testing results have indicated that the developed mix has superior mechanical properties over conventional high strength concrete mixes, such as an average compressive strength of 12 ksi at release and 18 ksi at 28 days.

The developed mix was applied to the design of double tee girders for short and medium span bridges. The standard PCI double tee girders (heavy section) were selected because of their cost effectiveness as well as ease and speed of production. Also, large 0.7 in. diameter prestressing strands were used to increase the flexure capacity of the girders and, consequently, their span-to-depth ratio. Several experimental investigations were carried out on two 51 ft long and 23.75 in. deep single tees (two halves of one double tee girder) to evaluate their flexural and shear capacity, in addition to the transfer and development length of 0.7 in diameter strands, and shear transfer between the UHPC girder and CIP concrete deck. Based on the results of these investigations, the following conclusions were made:

1. Current AASHTO LRFD specifications for flexure and shear design of bridge I-girders are applicable to UHPC bridge girders. Actual flexural and shear capacities compared very well with the predicted values.
2. Transfer length of 0.7 in. diameter strands with the investigated jacking stress (0.66 f_{pu}) in UHPC girders is significantly shorter than that predicted by the 2007 AASHTO LRFD Specifications.
3. Development length of 0.7 in. diameter strands in UHPC girders is conservatively predicted by the current 2007 AASHTO LRFD specifications. Accordingly, 0.7 in. diameter strands can be safely used in prestressing short span UHPC bridge girders.
4. Horizontal shear reinforcement is the primary contributor to the shear transfer between precast UHPC girder and CIP deck. The contribution of the contact surface between precast UHPC girder and CIP deck should be ignored due to the

difficulty of roughening the top surface of UHPC girders, until further investigation.

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