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Application of Ultra-High Performance Concrete to Bridge Girders

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16. Abstract
Ultra-High Performance Concrete (UHPC) 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 that produces a very tightly packed mix, extremely low water to powder ratio, and use of steel fibers. The unique strength and durability properties of UHPC make it an attractive material for precast prestressed bridge girder construction. However, commercial UHPC mixes currently available in the US market cost about 10 times the cost of conventional concrete mixes, in addition to the need for special mixing and curing procedures that are not convenient to most precasters. The general objective of this project is to promote the use of UHPC in bridge construction. The specific objectives include: 1) review the various UHPC mixes developed in North America, Europe, and Japan and compare them in terms of economics and performance characteristics; 2) develop non-proprietary UHPC mix that are optimized in terms of the total cost of production while providing a final compressive strength of at least 18 ksi; 3) evaluate the mechanical properties of the developed mixes; and 4) investigate the application of the developed mixes to standard precast prestressed concrete bridge I-girders. The developed mixes consist of type III cement, fine sand, class C fly ash, silica fume, high range water reducer, and water. Steel fibers are eliminated due to their high cost and Grade 80 ksi welded wire reinforcement (WWR) is used instead to substitute for the loss in the tensile/shear capacity. The results of the laboratory tests and the full-scale girder tests indicated that the developed mixes are attainable using practical and affordable mixing and curing procedures and their mechanical properties are superior to those of the mixes currently used in Nebraska.

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Abstract

Ultra-High Performance Concrete (UHPC) 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 that produces a very tightly packed mix, extremely low water to powder ratio, and use of steel fibers. The unique strength and durability properties of UHPC make it an attractive material for precast prestressed bridge girder construction. However, commercial UHPC mixes currently available in the US market cost about 10 times the cost of conventional concrete mixes, in addition to the need for special mixing and curing procedures that are not convenient to most precasters.

The general objective of this project was to promote the use of UHPC in bridge construction. The specific objectives included: 1) a review of the various UHPC mixes developed in North America, Europe, and Japan and a comparison of them in terms of economics and performance characteristics; 2) development of non-proprietary UHPC mix that was optimized in terms of the total cost of production while providing a final compressive strength of at least 18 ksi; 3) evaluation the mechanical properties of the developed mixes; and 4) investigate the application of the developed mixes to standard precast prestressed concrete bridge I-girders. The developed mixes consist of type III cement, fine sand, class C fly ash, silica fume, high range water reducer, and water. Steel fibers are eliminated due to their high cost and Grade 80 ksi welded wire reinforcement (WWR) was used instead to substitute for the loss in the tensile/shear capacity. The results of the laboratory tests and the full-scale girder tests indicated that the developed mixes are attainable using practical and affordable mixing and curing procedures and their mechanical properties are superior to those of the mixes currently used in Nebraska.

Chapter 1 Introduction

1.1 Problem Statement

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 basic constituents of the UHPC are not significantly different from those of the conventional concrete, including sand, type I/II Portland cement, quartz flour, silica fume, high-range water reducer (HRWR), and water. The enhanced strength and durability properties of UHPC are mainly due to optimized particle gradation that produces a very tightly packed, mix use of steel fibers, and an extremely low water to powder ratio. Currently, the only commercially available UHPC in the U.S. is marketed by Lafarge, Inc., since 2001, under the name Ductal®. This product is shipped to precasters in three separate components: preblended dry materials, steel fibers, and chemical admixtures. The cost of these components varies significantly based on their proportions, but is approximately \$750 to \$1,000/yd^3, which is over 10 times the cost of conventional concrete mixes. This is in addition to the high production cost due to longer mixing and curing operations (i.e. 45 mins mixing and 48 hrs intensive steam curing to achieve the expected 20 to 30 ksi strength). Moreover, typical UHPC mixes have delayed setting and need longer time to remove the product from the prestressing bed, which could double the production cost.

In spite of the unique strength and durability characteristics of UHPC, the extremely high material and production costs represent a serious obstacle towards its wide use in practical and economical bridge applications. Therefore, there is a vital need for research to investigate alternative mixes that are more economical and have comparable mechanical properties to the currently available products, in addition to adequate workability, practical mixing and curing, and sufficient early strength.

1.2 Objectives

The general objective of this research is to promote the use of UHPC in the construction of precast prestressed bridge girders in Nebraska. The specific objectives are to:

1) Develop an economical and practical UHPC mix(es) that has a target compressive strength of 18 ksi and performance characteristics superior to those of the mixes currently used in Nebraska.

2) Investigate the use of the developed UHPC mix(es) in developing an optimized section for prestressed bridge girders using the forms that are readily available to precast producers in Nebraska.

1.3 Report Outline

The remainder of this report is divided as follows.

Chapter 2 Literature Review: The literature review presents the development of HPC and UHPC mixes and their applications to bridge construction. The literature includes the research program led by the FHWA on the potential use of UHPC in bridge superstructure as well as the research projects conducted by other states, such as Iowa, Ohio, and Virginia.

Chapter 3 Developing NU UHPC Mixes: This chapter presents the various trial mixes developed to satisfy the workability, practicality, strength, and economy requirements. Local materials, such as fine sand, limestone, and Class C fly ash, were used to minimize material cost. Type III cement was used to achieve high early strength. Large quantities of silica fume, high range water reducer and water were used to satisfy the strength and workability requirements.

Chapter 4 Material Testing of NU UHPC: This chapter presents the results of the following material tests performed on the developed NU UHPC mixes:

1. Slump-Flow (ASTM C1611)
2. Compressive Strength (ASTM C39; 1, 3, 14, and 28-day strength)
3. Modulus of Elasticity (ASTM C469; 28-day modulus)
4. Split Cylinder Cracking Strength (ASTM C496)
5. Prism Flexure Cracking Strength (ASTM C78)
6. 28 Day Length Change (ASTM C157)
7. AASHTO Shrinkage Test (NCHRP Report 496; final mixes only)

Chapter 5 Applications of NU UHPC to Bridge I-Girders: This chapter presents the application of the two selected NU UHPC mixes to bridge I-girders. The first application was the shear testing of two AASHTO type II girders made of one of the recommended NU UHPC mixes. The second application was the flexural testing of NU900 girder made of an alternative NU UHPC mix. Testing results were compared against those obtained from testing similar girders made of conventional concrete.

Chapter 6 Design and Production Recommendations: This chapter presents design, detailing, and production recommendations for using the developed NU UHPC mixes in bridge girders. These recommendations were developed based on the test results and the experience gained from the applications presented in Chapter 5.

Chapter 2 Literature Review

2.1 High Performance Concrete

According to the American Concrete Institute (ACI), high performance concrete (HPC) is defined as concrete that has special combination of characteristics and uniformity requirements, which cannot be achieved using conventional constituents, mixing, and curing procedures. The characteristics and requirements of HPC are (Goodspeed et al., 2006):

1. Ease of placement (good filling and passing ability).
2. High early strength.
3. Long-term mechanical properties.
4. Low Permeability.
5. Volume stability.
6. Long life in severe environments.

In 1987, Congress initiated a five-year Strategic Highway Research Program (SHRP) to investigate different concrete products to improve the standards, maintenance and rehabilitation activities of the nation's highways and bridges. In order to set a definition for HPC, the SHRP program specified the following criteria for HPC (Zia et al., 1991):

1. A maximum water-to-powder ratio of 0.35.
2. A minimum durability factor of 80%, as determined by ASTM C666.
3. A minimum strength of:
    - 3000 psi at age of 4 hours.
    - 5000 psi at age of 24 hours.
    - 10000 psi at age of 28 days.

In 1993, the Federal Highway Administration (FHWA) initiated a national program to introduce HPC to bridge construction. The FHWA program included the construction of HPC demonstration bridges in all FHWA regions. The technology and results of HPC bridge construction were presented at showcase workshops. The intent of this program was to show the different states how they can benefit from the use of HPC in bridge construction. A complete list of the states participating in the FHWA HPC Bridge Showcase program as of February 1999 can be found in (Rabbat et al. 1999).

The construction of the first HPC Bridge in the State of Nebraska began in the summer of 1995. The 225 ft. long bridge had three spans of 75 ft each, and utilized seven precast/prestressed HPC girders per span. The project specifications utilized one performance characteristic to define HPC for girders and two HPC performance characteristics for the bridge deck. The girders' compressive strength was specified as 12,000 psi at 56-days. Deck strength was specified as 8,000 psi at 56-days with a chloride penetration of less than 1800 coulombs at 56 days (measured in accordance with AASHTO T 277). More details on Nebraska's first HPC Bridge are available in Beacham (1999).

According to the FHWA, HPC is defined as "A concrete made with appropriate materials combined according to a selected mix design; properly mixed, transported, placed, consolidated and cured so that the resulting concrete will give excellent performance in the structure in which it is placed, in the environment to which it is exposed and with the loads to which it will be subject for its design life" (Forster, 2006).

The FHWA selected a set of performance characteristics to quantify its HPC definition. These include four structural characteristics: compressive strength, modulus of elasticity, shrinkage, and creep, in addition to durability conditions, such as: freeze-thaw resistance, scaling resistance, abrasion resistance, chloride ion penetration, alkali-silica reactivity, and sulfate resistance. Four grades of performance are specified for each of the afore-mentioned characteristics, where higher grade is assigned to higher performance. The structural and durability performance and grade of a HPC mix is selected according to the intended use of the mix. Grades of performance of HPC are shown in table 2.1.

Table 2.1 FHWA Performance Grade Guidelines (Goodspeed et al. 2006)

Performance Characteristic	Standard Test Method	FHWA HPC Performance Grade			
		1	2	3	4
Freeze/Thaw Durability (x = relative dynamic modulus of elasticity after 300 cycles)	AASHTO T 161 ASTM C 666A	60% ≤ x ≤ 80%	80% ≤ x	-	-
Scaling Resistance (x = visual rating of the surface after 50 cycles)	ASTM C 672	x = 4, 5	x = 2, 3	x = 0, 1	-
Abrasion Resistance (x = average depth of wear in mm)	ASTM C 944	2.0 > x ≥ 1.0	1.0 > x ≥ 0.5	0.5 > x	-
Chloride Permeability (x = permeability in coulombs)	AASHTO T 277 ASTM C 1202	3000 ≥ x ≥ 2000	2000 ≥ x > 800	800 ≥ x	-
Compressive Strength (x = compressive strength)	AASHTO T 22 ASTM C 39	41 ≤ x < 55 Mpa (6 ≤ x < 8 ksi)	55 ≤ x < 69 Mpa (8 ≤ x < 10 ksi)	69 ≤ x < 97 Mpa (10 ≤ x < 14 ksi)	x ≥ 97 Mpa (x ≥ 17 ksi)
Modulus of Elasticity (x = modulus of elasticity)	ASTM C 469	24 ≤ x < 40 Gpa (4 ≤ x < 6 *10^6 psi)	40 ≤ x < 50 Gpa (6 ≤ x < 7.5 *10^6 psi)	x ≥ 50 Gpa (x ≥ 7.5 *10^6 psi)	-
Shrinkage (x = microstrain)	ASTM C 157	800 > x ≥ 600	600 > x ≥ 400	400 > x	-
Creep (x = microstrain / pressure unit)	ASTM C 512	0	0	0	0

To date, several research programs have focused on the development of economic self-consolidating HPC mixes. The National Cooperative Highway Research Program (NCHRP) Report 579 presented several HPC mixes developed at Wiss, Janney, Elstner Associates, Inc. (WJE). The mixes presented used aggregates supplied from a precaster (Prestress Engineering Cooperation) and trap rock aggregate available from Wisconsin. After several trial mixes, a concrete compressive strength of 17.8 ksi was achieved. Tables 2.2 and 2.3 present summaries of the developed mixes and their strength properties, respectively.

Table 2.2 NCHRP Report 579 Concrete Mix Designs

Material (lbs/yd^3)	Mix					
	1 & 2	3 & 4	5 & 6	7 & 8	9	10
Type I /II Cement	-	-	1050	-	-	1050
Type III Cement	750	1030	-	1030	700	-
Fly Ash	-	-	-	-	-	-
Silica Fume	-	125	150	125	-	150
Water	210	300	264	300	280	264
Sand	1328	777	858	777	1180	858
3/4 " Aggregate	1880	-	-	-	1786	-
1/2" Aggregate	-	1820	-	1820	-	-
3/8" Aggregate	-	-	1820	-	-	1820
Retarder (100XR)	-	-	4 oz/100lbs cwt	20 oz/yd^3	-	4 oz/100lbs cwt
Super Plasticizer	-	As Needed	15-18 oz/100lbs cwt	As Needed	175 oz/yd^3	15-18 oz/100lbs cwt
w/c	0.28	0.24	0.25	0.24	0.4	0.25

Table 2.3 NCHRP Report 579 Concrete Testing Results

Mix #	Compressive Strength (ksi)	Splitting Strength (psi)	Modulus of Rupture (psi)
1	12.1	867	991
2	12.6	811	991
3	15.9	766	1090
4	16.3	766	1090
5	17.8	894	1190
6	12.7	823	1190
7	12.5	706	720
8	13.3	706	720
9	9.6	686	1080
10	10.6	765	1180

2.2 Ultra High Performance Concrete

In the mid-1990s, researchers in France developed a new generation of HPC called Reactive Powder Concrete. The unique characteristics of this concrete are mainly due to the selection and proportioning of mix constituents that achieve an optimized packing order for the granular mixture. The optimized particle gradation results in a low void ratio and higher strength. The largest granular material in a reactive powder mix is fine sand, with a particle size ranging from 150 μm to 600 μm. Cement particles have the second largest size in the mix, with a nominal size of 15 μm. Quartz flour has a nominal diameter of 10 μm and Silica fume is the finest particle in the mix, with a nominal size of 1 μm. Supplementary cementitious materials such as silica fume and quartz flour are used to increase the concrete performance characteristics. Silica fume, as a very reactive pozzolanic, reacts with the calcium hydroxide resulting from Portland cement hydration. This reaction forms additional binder material called calcium silicate hydrate. This additional binder improves the hardened concrete properties. In addition, silica fume increases the cohesion of fresh concrete, which reduces segregation and bleeding. The extremely fine size of silica fume particles minimizes the voids in hardened concrete, which results in reduced permeability and enhanced mechanical properties. Additional properties of silica fume are available in the Silica Fume User's Manual.

The exceptional properties of reactive powder concrete, which is referred to later as Ultra High Performance Concrete (UHPC), are also due to the extremely low water to-powder ratio and the use of steel fibers. Tables 2.4 and 2.5 show the constituents and material properties of a typical UHPC.

Table 2.4 Typical UHPC Composition (Greybeal, 2003)

Material	lbs/yd^3	% by wt.
Portland Cement (15 μm)	1200	28.7
Silica Fume (~1 μm)	390	9.3
Quartz Flour (10 μm)	355	8.4
Sand (150 to 600 μm)	1720	40.8
Steel Fibers (0.5" long, 8 mm Ø)	263	6.2
High-Range Water Reducer	51.8	1.2
Accelerator/Corrosion Inhibitor	50.5	1.2
Water	184	4.4

Table 2.5 Typical UHPC Properties (Perry, 2005)

Property	Mean Values
Compressive Strength	20,000 - 30,000 psi
Flexural Strength	3,500 - 6,000 psi
Young's Modulus	8 - 8.5 x 10^6
Freeze/Thaw (300 cycles)	1
Salt Scaling (loss of residue)	< 0.0025 lb/ft^2
Abrasion (relative volume loss index)	1.2
Oxygen Permeability	< 10^{-19}/ft
CI Permeability (total load)	< 10
Carbonation Depth	< 0.02 in
Chloride Ion Diffusion (CI)	0.02 x 10^{-11}ft^2/s

The Association Francaise de Genie Civil (AFGC) *Interim Recommendations for Ultra-High Performance Fibre-Reinforced Concretes (2002)* defines UHPC as a material with a characteristic compressive strength more than 20 ksi (150 MPa), and contains steel fibers resulting in ductile behavior. The Japan Society of Civil Engineers (JSCE) *Recommendations for Design and Construction of Ultra-High Strength Fiber-Reinforced Concrete Structures (2006)* defines the UHPC as "A type of cementitious composite reinforced by fiber with characteristic values in excess of 150 MPa in compressive strength, 5 N/mm^2 in first cracking strength". A wide range of proprietary UHPC mixes are available in the U.S. and international market. Examples of proprietary mixes are Beton Special Industrial (BSI) concrete developed by Eiffage; Cemtec by LCPC; and Ductal® concrete resulting from a joint research by Bouygues, LaFarge, and Rhodia. Ductal® concrete, marketed by LaFarge and Bouygues, is the only patented UHPC product in the US market.

Quartz flour has a particle size larger than silica fume and smaller than cement and is used as a supplementary cementitious material, to improve the mix packing order in UHPC. Due to the low water-to-powder ratio of UHPC, a significant portion of Portland cement particles remain un-hydrated. These un-hydrated cement particles remain inert within the mix, and act like fine aggregate particles. In a relevant study, Ma and Schneider (2002) incrementally replaced portions of the cement with quartz flour of equivalent volume. The replacement of cement portions, up to 30% by weight, did not affect the final strength of the mix. The mixes developed in this study are shown table 2.6.

Table 2.6 Ma and Schneider (2002) UHPC Mixes with Compressive Strength Results

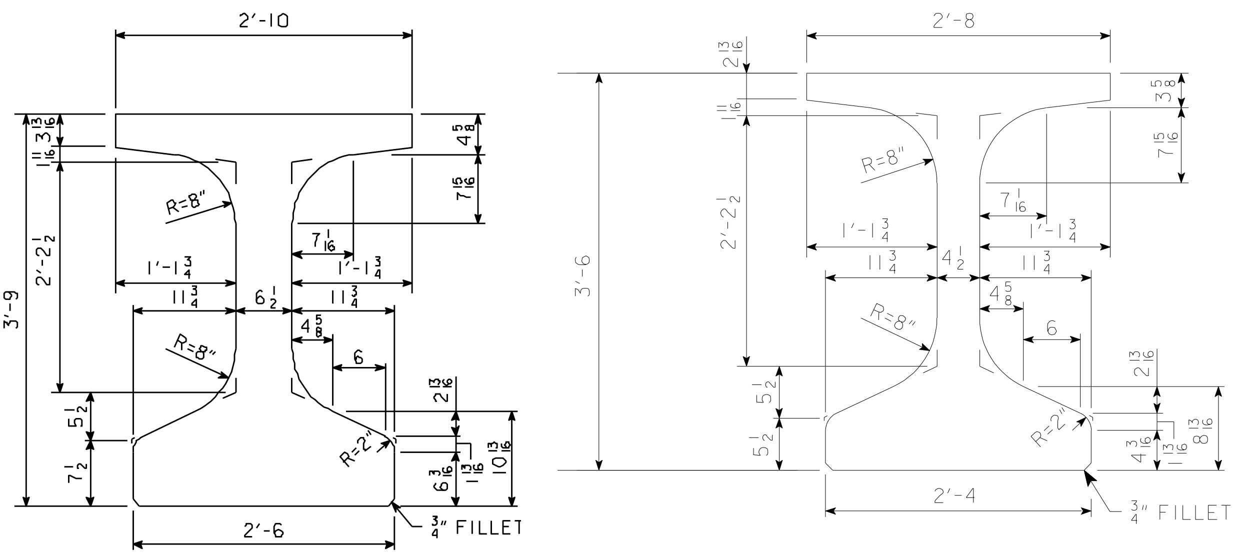
Constituent (lbs)	Mix #1	Mix #2	Mix #3	Mix #4	Mix #5	Mix #6
Aggregate	**1718**	**2250**	**2474**	**2474**	**2491**	**2669**
Sand	1718	671	742.0	742	747	801
Large Aggregate	0	1578	1731.9	1732	1744	1868
Cementitious Material	**1939**	**1776**	**1573**	**1573**	**1574**	**1398**
Cementitious Material	1121	1027	910.6	911	911	809
Silica Fume	337	309	273.2	273	164	243
Quartz Flour	481	440	389.5	390	499	346
Water and Water Reducer	**339**	**310**	**303**	**303**	**303**	**298**
Water/Powder	0.155	0.155	0.175	0.175	0.175	0.195
HRWR	39	36	28.3	28	28	25
Compressive Strength (ksi)	22.6	22.6	21.9	21.9	21.8	21.6

2.3 UHPC Bridge Girders

North America's first full scale vehicle bridge designed using UHPC was built by the FHWA at the Turner-Fairbank Highway Research Center in McLean, Virginia in 2004. The bridge has a single lane at 16 ft (4.9 m) wide and is 69 ft (21 m) long with only a 33.5 in (0.84 m) girder depth. The girders used were Pi-Girders developed by Massachusetts Institute of Technology (MIT) and shown in figure 2.1. The girders were designed using simple 2-D flexural analysis as well as a more complex finite element analysis to check for local behavior. To ensure the bridge is behaving as predicted, it is not open to public traffic and is to be frequently tested. Details on this bridge can be found in Greybeal (2005).

Figure 2.1 Pi-Girder Section

Another UHPC bridge was built in 2006 in Wapello County, Iowa, called The Mars Hill Bridge. The bridge is 113 ft long and 24.5 ft wide, and uses three 45 in. Iowa bulb tees. The Iowa standard bulb tee was modified, as shown in figure 2.2, because of the high quality concrete being used. The girders then ended up with 4.5 in. thick web (from 6.5 in.), a 5.5 in. deep bottom flange (from 7.5 in.) and 2.75 in. deep top flange (from 5.75 in.).



(a) (b)

Figure 2.2 (a) Standard Iowa Bulb Tess (b) Modified section for UHPC Girder

The Mars Hill Bridge, shown in figure 2.3, while not unique in appearance, uses UHPC to eliminate the need for mild steel reinforcement. This project was a joint effort of the FHWA, Iowa DOT, Iowa State University and Lafarge North America. The purpose of the bridge was to help develop specifications for the design of UHPC girders. An integral part of the project was the full-scale laboratory testing of the girders to evaluate the shear and flexure capacities of the new design. Testing results have indicated that the shear and flexural strength of UHPC outperformed the calculations. The bridge was opened to traffic in February of 2006 and was to be monitored for at least two years. (Moore et al. 2006).

Figure 2.3 Mars Hill Bridge, Wapello County, Iowa

Design plans were finalized in the spring of 2008 for a second full scale bridge in Buchanan County, Iowa. Designed by Iowa DOT and Iowa State University, the bridge is a total of 115 ft long, including one simple span of 50 ft using second generation UHPC Pi-girders. After full scale tests were performed by the FHWA, it was found that the original UHPC Pi-girder's flange does not have the required transverse strength, nor did the transverse connections between adjacent girder flanges behave acceptably. After consideration of possible improvements, the following changes were made: fillets were added to the stems to improve mix flow; flange thicknesses were increased to 4 $^{1}/_{8}$ in. to meet service requirements; web spacing was reduced to provide balance; and transverse mild steel reinforcement was added to the flange due to the lack of supporting test data. Figures 2.4 and 2.5 show the cross sections of the original and second generation of UHPC Pi-girders, respectively. Construction on Iowa's second UHPC bridge is to begin in the fall of 2008 (Keierleber et al. 2008).

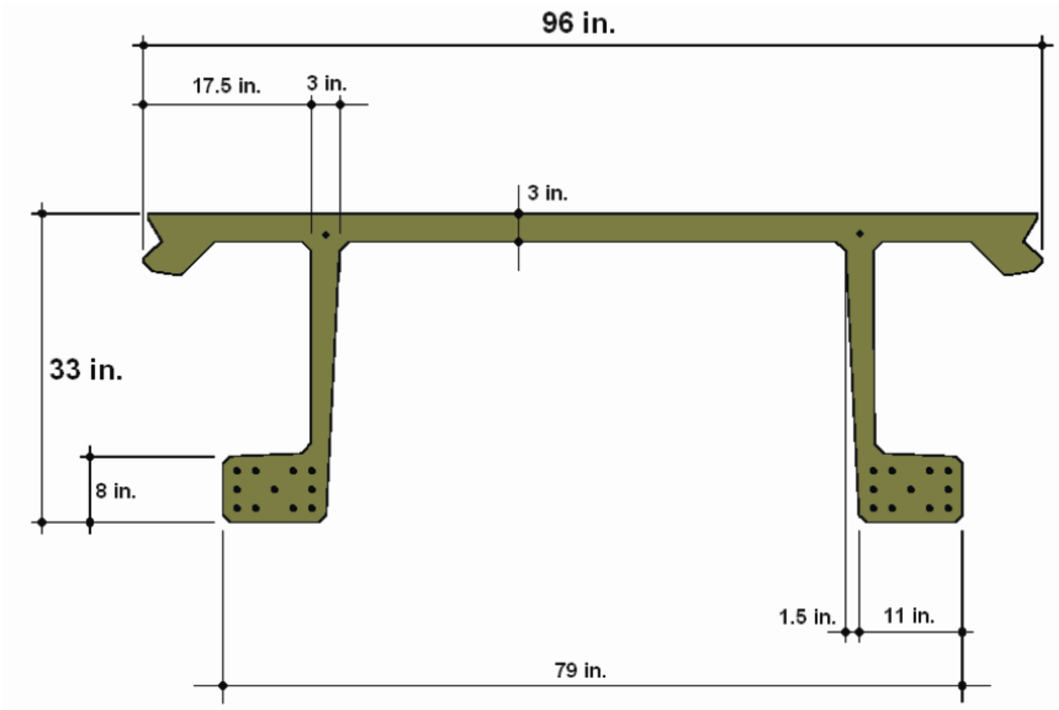


Figure 2.4 Original UHPC Pi-Girder Cross Section

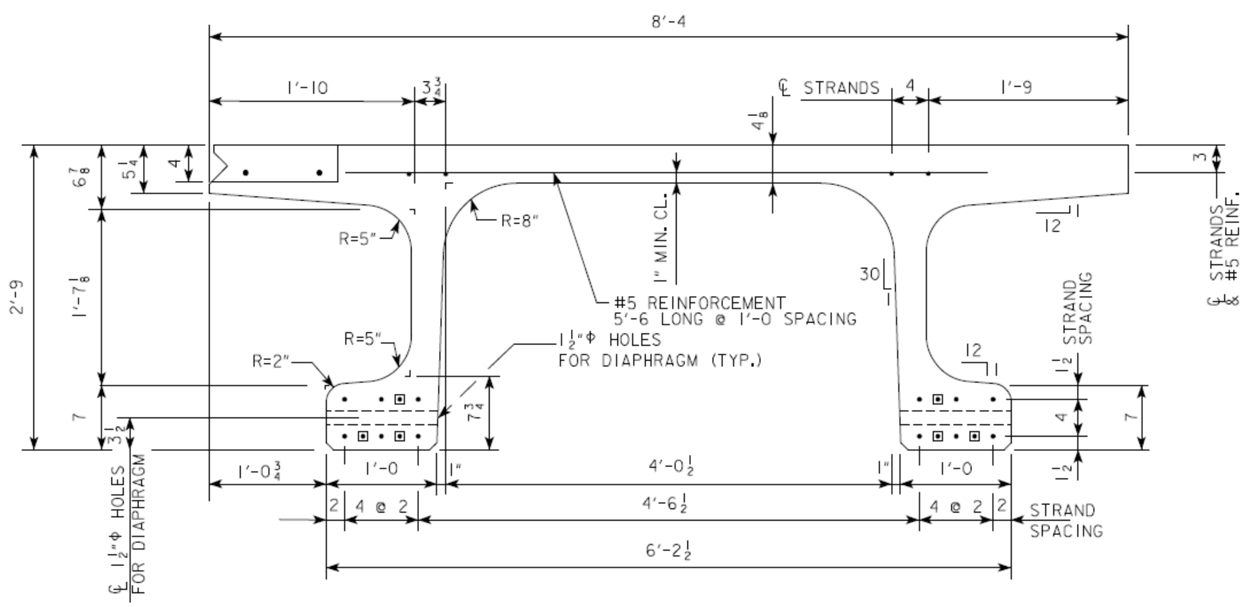


Figure 2.5 Second Generation UHPC Pi-Girder Cross Section

Chapter 3 Developing NU UHPC Mixes

3.1 Background

The first attempts to develop UHPC mixes at the University of Nebraska were done in 2006 using a 1 ft^3 Hobart food processing mixer equipped with a ¾ horsepower motor. This was necessary because a typical drum mixer was found to be inefficient at mixing the constituents with such low water-to-powder ratios. Table 3.1 shows the constituents for three of the NU UHPC mixes developed in 2006. The estimated cost per cubic yard was based on typical material costs in Nebraska including $90/ton for Portland cement, $600/ton for silica fume, $15/ton for Class C fly ash, $10/ton for fine sand, and $20/gallon for the high-range water-reducer used (i.e. Glenium 3000NS). Due to the small batch sizes, only compressive strength test cylinders were taken. No additional testing was performed to evaluate other mechanical or durability properties for the developed mixes. Figure 3.1 shows the results of the compressive strength versus time for three of the developed mixes.

Table 3.1 Initial NU UHPC Mix Constituents **(**Kleymann et al., 2006)

Material (lb/yd^3)	**NU UHPC # 2**	**NU UHPC #3**	**NU UHPC #7**
Fine Sand	1758	1716	1730
Cement I/II	1227	1217	1207
C Fly Ash	363	360	372
Silica Fume	399	395	382
HRWR	81	107	86
Water	204	202	221
w/c ratio	0.125	0.132	0.137
Cost per yd^3	$380	$441	$385

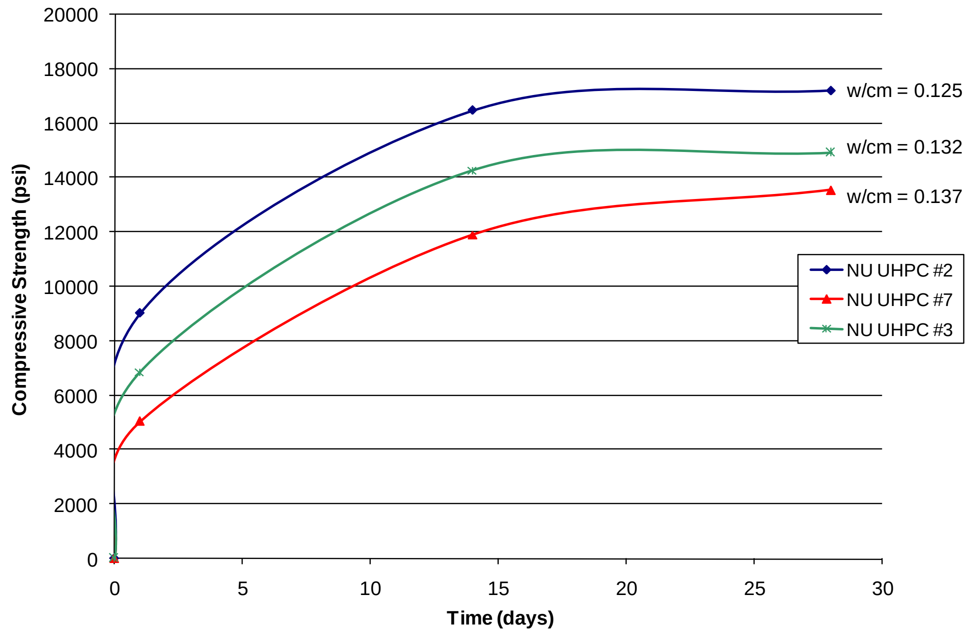


Figure 3.1 Compressive Strength of initial NU UHPC versus time (Kleymann et al., 2006)

In these initial attempts, the focus was to specify batching and mixing procedures using a maximum water-to-powder ratio of 0.2. First, the gradual addition of water and HRWR to the preblended granular materials produced concrete with large clumps. A significant amount of additional water was required to produce workable mixes, which had a negative impact on the compressive strength of the mixes. An alternative batching and mixing procedure was successfully achieved using following steps:

1. Dry blend all granular materials in the concrete mix. Including all cement, silica fume, Class C fly ash, and aggregates;
2. Place preblended granular materials in a separate container;
3. Add all water and ½ HRWR amount to the mixer;
4. Gradually add pre-blended granular material to the mixer;

5. The remaining amount of HRWR is gradually added to the mix over a period of 1 minute;

6. Mixing continues until sufficient mix workability is achieved (approx. 15 – 20 mins).

However, this batching and mixing procedure was found to be labor intensive because storing the preblended material in separate containers and adding them gradually to the water and HRWR is a time-consuming operation, especially with large mixes. Also, a concrete vibrator was needed to consolidate the concrete in the cylinders because of inadequate flowability.

In order to eliminate the problems associated with small size batches and inadequate mixing power, a high energy paddle mixer was used. The Imer Morterman 750 series mixer shown in figure 3.2 has a 5.5 horsepower motor and a batch capacity up to 18 ft^3. This mixer was used in all the mixes developed within this project.

Figure 3.2 Imer Mortar Mixer

3.2 Materials Used

Several types of aggregates were investigated to select the most appropriate local materials for developing NU UHPC mixes. Fine sand (#10 sand), overlay sand, block sand, 47B sand and gravel, and C33 sand (4110 sand) were supplied from Ready Mix Company, Omaha, Nebraska. Quarter inch limestone was supplied from Martin Marietta Aggregates, Papillion, Nebraska. Pre-washed half inch limestone (BRS) was obtained from Concrete Industries, Inc., Lincoln, Nebraska. Samples of all aggregates were oven dried and a sieve analysis was performed using standard sieve sizes. Results of the sieve analysis for fine and coarse aggregates are shown in figures 3.3 and 3.4, respectively.

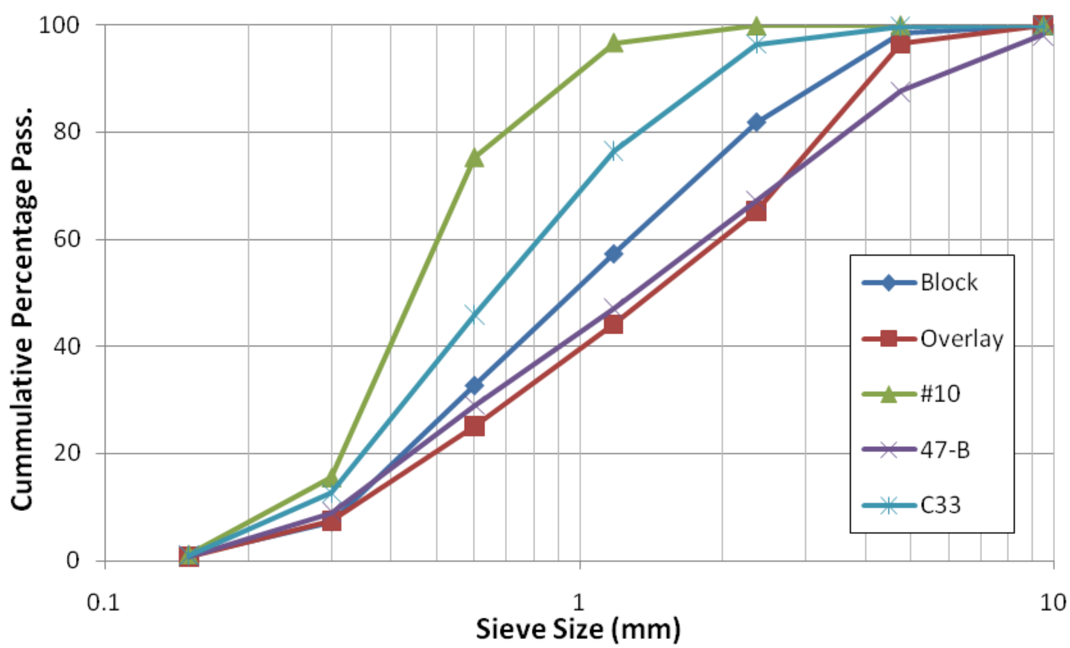


Figure 3.3 Fine Aggregate Sieve Analysis

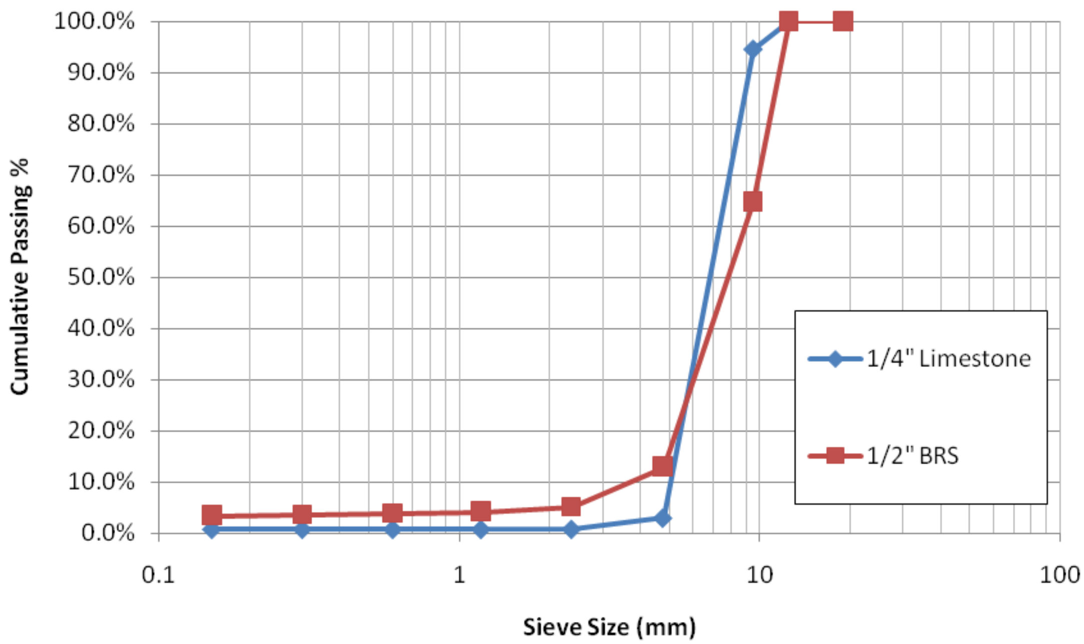


Figure 3.4 Coarse Aggregate Sieve Analysis

Also, several types of high-range water reducers (HRWR) were used, such as Glenium 3000NS, Glenium 3030, and Glenium 7700 supplied by BASF Construction Chemicals, LLC, Omaha, Nebraska; and Chryso Fluid Premia 150 supplied by Chryso Company, Charlestown, Indiana. Chryso Fluid Premia 150 was eventually selected for the final mixes for its efficiency and economy.

3.3 NU UHPC Group #1 Mixes

The purpose of the Group #1 mixes, listed in table 3.2, was to define practical batching and mixing procedures. Mixes #1 and #2 were batched and mixed according to procedures presented earlier, but mix flowability was not adequate (i.e. average spread diameters less than 22 in.). In mixes 3 and 4, dry mixing of granular materials took place for 2 minutes, then water and HRWR were gradually added. These procedures resulted in adequate flowability; however, mixing time exceeded 30 minutes, which is impractical. Compressive strengths of the four mixes at 1, 3, 7, 14, and 28 days are shown in figure 3.5.

Table 3.2 Design and Cost of Group # 1 Mixes

Mix Number	Mix #1	Mix #2	Mix #3	Mix #4
Aggregate	**1758**	**1716**	**2474**	**2029**
#10 Fine Sand	100%	100%	70%	100%
1/4" Limestone	0%	0%	30%	0%
Cementitious Material	**1989**	**1972**	**1573**	**1567**
Cement Type I/II	62%	62%	58%	67%
Class C Fly Ash	18%	18%	25%	25%
Silica Fume	20%	20%	17%	8%
W/CM Ratio	**0.161**	**0.172**	**0.24**	**0.184**
Water	251	250	330	288
Glenium HRWR	127	162	85	34
Cost (USD/yd^3)	**339**	**379**	**239**	**139**

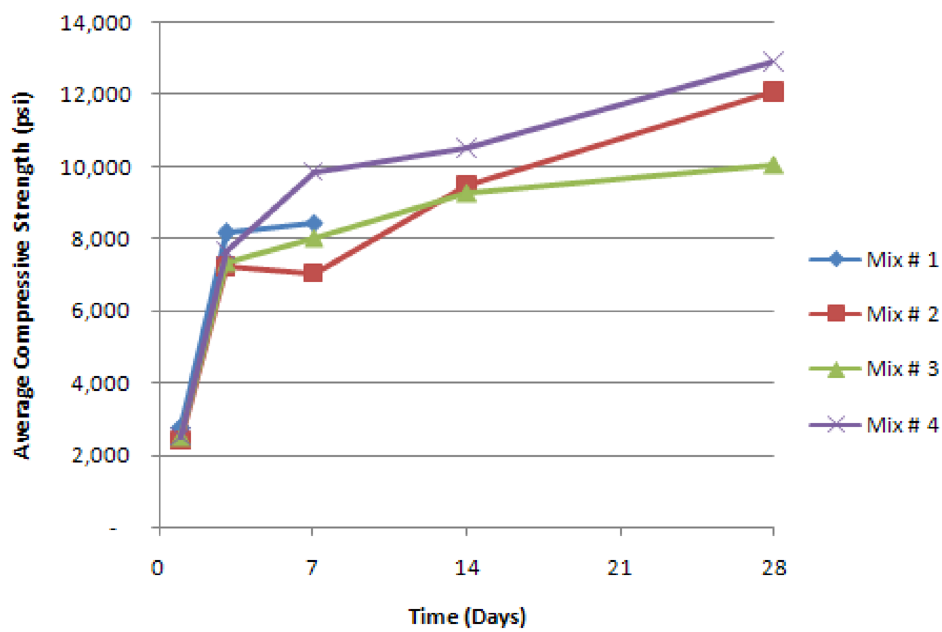


Figure 3.5 Compressive Strength of Group # 1Mixes versus Time

Figure 3.5 indicates that high early compressive strength was not achieved in any of these mixes. In addition, none of the mixes reached a compressive strength of 13 ksi at 28 days. These mixes were also uneconomical in terms of material cost and production cost due to long mixing time.

3.4 NU UHPC Group #2 Mixes

Two mixes were tried in Group #2 to evaluate a slightly different mixing procedures from Group # 1. The constituents of the two mixes #5 and #6 are listed in table 3.3. In these mixes, all granular materials were mixed for 2 minutes, all the water and half the HRWR was added, mixing continued for 15 minutes before adding the other half of the HRWR, then mixing was resumed for another 3 minutes (total of 20 minutes). None of the two mixes demonstrated sufficient filling ability using these procedures. Additional quantities of HRWR were added and mixing continued for more than 35 minutes. Because of inadequate flowability, the cylinders had excessive voids, which resulted in very low strength results. All cylinders were disposed of after the 24-hours test. Because of the lack of flowability, the research team decided to use a different type of HRWR to achieve the required flowability without significant effect on the economy of the mix.

Table 3.3 Design and Cost of Group #2 Mixes

Mix Number	Mix #5	Mix #6
Aggregate	**2070**	**1758**
#10 Fine Sand	100%	100%
1/4" Limestone	0%	0%
Cementitious Material	**1570**	**1960**
Cement (Type)	60% (I/II)	63% (III)
Class C Fly Ash	22%	19%
Silica Fume	18%	18%
W/CM Ratio	**0.172**	**0.183**
Water	270	294
Glenium HRWR	40	117
Cost (USD/yd^3)	**187**	**310**

3.5 NU UHPC Group #3 Mixes

The constituents for the Group #3 mixes were similar to those of Group # 1 and Group # 2 mixes except for the type of cement and HRWR used. In this group, type III cement was used to increase the early strength and Chryso Premia 150 was used to achieve a minimum spread of 22 in. and reducing the total material cost (Chryso costs $10 per gallon). Group #3 mix designs are shown in table 3.4. The average spread diameters for all the mixes ranged from 23 in. to 25 in. with no visual bleeding or segregation in any of the mixes. Compressive strength was tested at ages 1, 3, 7, 14, and 28 days for each mix. Cylinders poured for compressive strength tests were immediately covered and placed in the moisture chamber for curing (72^o F at 95% humidity). The molds were stripped after 24 hrs and the concrete cylinders were returned to the moisture room, until they reached the appropriate age. Cylinder ends were ground using an electric concrete saw with a leveling device and neoprene pads were used to distribute load on the top and bottom faces of the cylinder. Compressive strength tests for cylinders were performed according to ASTM C39. Figure 3.6 shows the compressive strength of the five mixes versus time. Figure 3.6 indicates that mix #11 resulted in a 1-day compressive strength of 11 ksi and a 28-day strength of 16 ksi using moisture curing. This mix was identified as one of the successful mixes which was chosen for further testing.

Table 3.4 Design and Cost of Group #3 Mixes

Mix Number	Mix #7	Mix #8	Mix #9	Mix #10	Mix #11
Aggregate	**2193**	**2070**	**2070**	**2070**	**2070**
#10 Fine Sand	100%	70%	70%	70%	70%
1/4" Limestone	0%	30%	30%	30%	30%
Cementitious Material	**1600**	**1600**	**1600**	**1600**	**1600**
Cement Type III	65%	65%	65%	60%	70%
Class C Fly Ash	20%	20%	15%	20%	15%
Silica Fume	15%	15%	20%	20%	15%
W/CM Ratio	**0.177**	**0.173**	**0.168**	**0.182**	**0.173**
Water	243	240	225	248	227
Chryso HRWR	72.5	68	80	78	68
Cost (USD/yd^3)	**227**	**210**	**247**	**242**	**224**

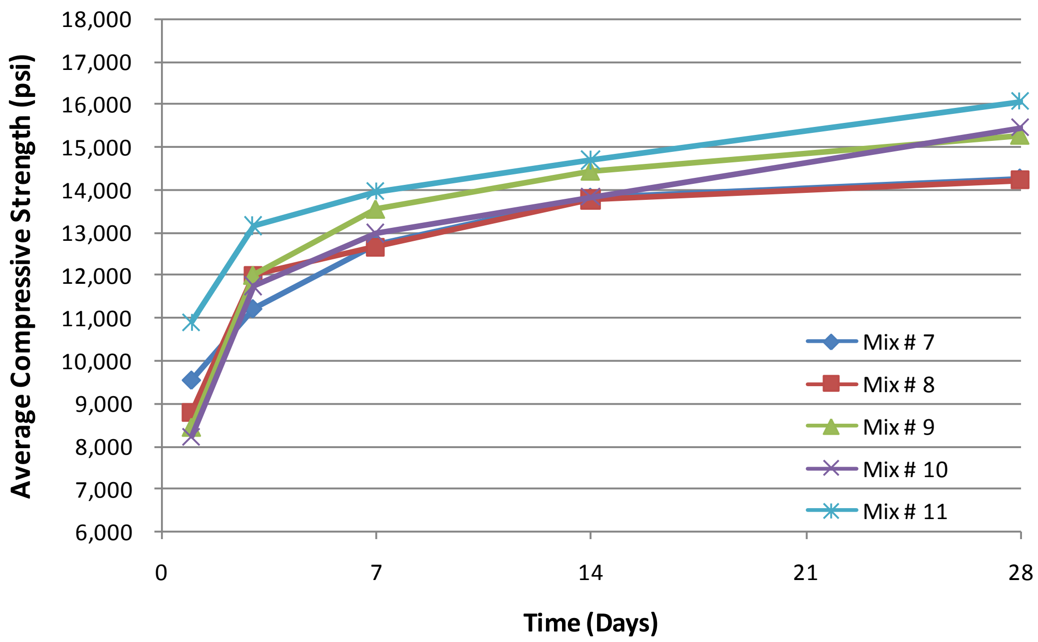


Figure 3.6 Compressive Strength of Group # 3 Mixes versus Time

3.6 NU UHPC Group #4 Mixes

Due to the high material cost of Group #3 mixes, several trial mixes were developed to lower the total material cost (Group #4 mixes). Lower quantities of cementitious and supplementary cementitious material as well as smaller dosages of HRWR were used. The first mix of this group, mix #12, was performed using type I/II cement in order to adequately gauge the necessity for type III cement. This mix performed poorly on day 1 which prompted the mix to be redone with type III cement as mix #13. Table 3.5 lists the final seven mixes developed within Group #4 along with their material cost (Akhnoukh, 2008). It should be noted that mix repeatability was investigated through developing two identical mixes #15 and #17.

Table 3.5 Design and Cost of Group #4 Mixes

Mix Number	Mix #13	Mix #14	Mix #15	Mix #16	Mix #17	Mix #18	Mix #19
Aggregate	**2434**	**2434**	**2434**	**2434**	**2434**	**2075**	**2468**
#10 Fine Sand	100%	100%	100%	35%	100%	35%	35%
C33 Sand	0%	0%	0%	35%	0%	35%	35%
1/2" BRS	0%	0%	0%	30%	0%	30%	30%
Cementitious Material	**1300**	**1300**	**1300**	**1300**	**1300**	**1600**	**1300**
Cement Type III	80%	80%	80%	80%	80%	70%	80%
Class C Fly Ash	10%	10%	10%	10%	10%	15%	10%
Silica Fume	10%	10%	10%	10%	10%	15%	10%
W/CM Ratio	**0.23**	**0.2**	**0.23**	**0.23**	**0.23**	**0.19**	**0.21**
Water	261	230	284	284	284	278	235
Chryso HRWR	54	41	27	23	27	27	38
Cost (USD/yd^3)	**160**	**145**	**130**	**127**	**130**	**165**	**144**

Figure 3.7 shows the final seven Group #4 mixes compressive strength results at days 1 and 3. A 1-day compressive strength of 9 ksi and a 3-day compressive strength of 11 ksi were set as acceptance criteria, in addition to sufficient flowability. Based on the day 1 and day 3 compressive strength results, mixes #14 and #18 were found to be acceptable and were selected for further mix modification and material testing. It is important to note that mix #18 is a low-cost variation of mix #11, of Group #3, with a higher w/c ratio and lower HRWR dosage. Both of the two chosen mixes have a maximum water-to-power ratio of 0.2. This ratio was considered to be the upper limit for the development of subsequent NU UHPC mixes. The repeatability investigation of mixes #15 and #17 showed that their day 1 and day 3 compressive strengths were very similar.

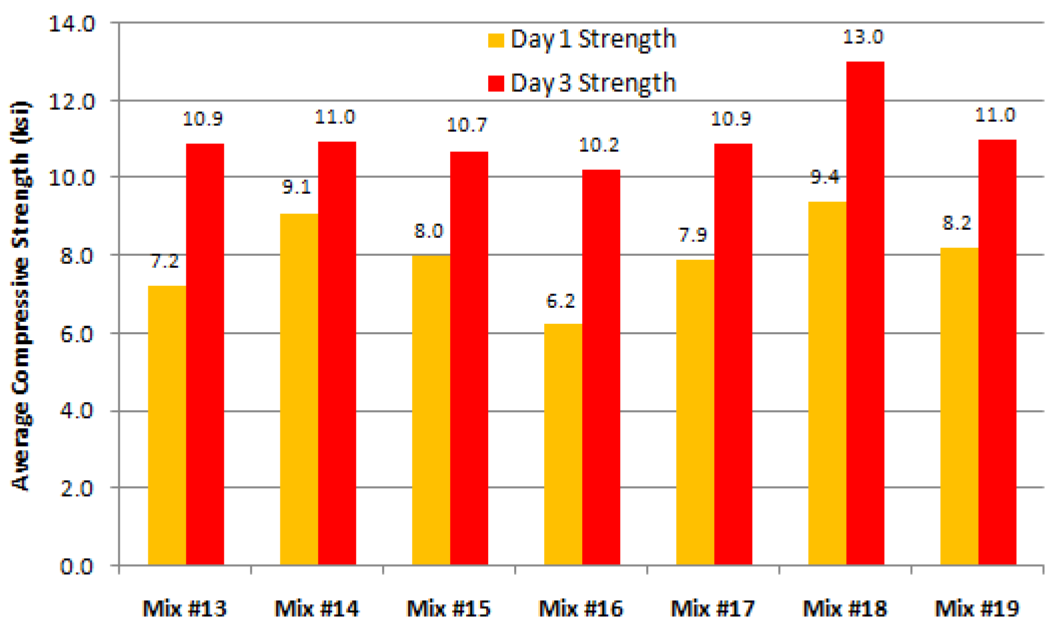


Figure 3.7 Day 1 and Day 3 Compressive Strength of Group #4 Mixes

Chapter 4 Material Testing of NU UHPC

Based on the results of the 19 trial mixes presented in Chapter 3, five mixes were developed as candidates for further material testing. These mixes are basically variations of the mixes #11 and #14, which were considered the best mixes in terms of early strength, final strength, and material cost. Table 4.1 lists the NU UHPC mixes developed for further material testing and their corresponding material cost. This chapter presents the details and results of the material tests performed to investigate the various properties of the developed mixes, such as flowability, compressive strength, flexure strength, splitting tensile strength, modulus of elasticity, length change, and shrinkage losses. All NU UHPC mixes listed in table 4.1 were developed in large quantities (4.5 ft^3) to allow making the specimens required for the tests listed in table 4.2. It should be noted that additional tests might be needed to investigate the durability properties of the developed mixes, such as chloride ion penetration, freeze and thaw resistance, alkali silica reactivity, and wet and dry resistance, before being applied to actual projects.

Table 4.1 Design and Cost of the NU UHPC mixes chosen for Material Testing

Material Type	NU UHPC #1	2/21/2008	NU UHPC #2	3/13/2008	NU UHPC #3	3/20/2008	NU UHPC #4	4/7/2008	NU UHPC #5	4/28/2008
	Weight (lb/yd^3)	Material Cost ($)	Weight (lb/yd^3)	Material Cost ($)	Weight (lb/yd^3)	Material Cost ($)	Weight (lb/yd^3)	Material Cost ($)	Weight (lb/yd^3)	Material Cost ($)
#10 Sand	2255	$ 11.3	2428	$ 12.1	1580	$ 7.9	2255	$ 11.3	1580	$ 7.9
1/4" BRS	0	$ -	0	$ -	672	$ 5.0	0	$ -	672.3	$ 5.0
Cement III	1050	$ 47.3	1040	$ 46.8	1050	$ 47.3	1120	$ 50.4	1050	$ 47.3
C Fly Ash	300	$ 2.3	130	$ 1.0	300	$ 2.3	240	$ 1.8	300	$ 2.3
Silica Fume	150	$ 45.0	130	$ 39.0	150	$ 45.0	240	$ 72.0	150	$ 45.0
Chryso	61.9	$ 69.6	35.4	$ 39.8	61.9	$ 69.6	70.8	$ 79.6	54.0	$ 60.7
Agg. Water	22.6	$ -	24.3	$ -	22.5	$ -	22.5	$ -	38.0	$ -
Free Water	225.0	$ -	260.0	$ -	240.0	$ -	240.0	$ -	227.0	$ -
W/C ratio	**0.165**	**$ 175.3**	**0.219**	**$ 138.7**	**0.175**	**$ 177.0**	**0.164**	**$ 215.0**	**0.177**	**$ 168.1**

Table 4.2 Material Tests of the Chosen NU UHPC Mixes

Test	Specs.	Specimens	Number	Volume	Total
Flowability	ASTM C1611	Slump Cone 4x8x12	1	0.20	0.20
Compressive Strength	ASTM C39	Cylinders 4x8	18	0.06	1.05
Splitting Strength	ASTM C496	Cylinders 4x8	3	0.06	0.17
Modulus of Elasticity	ASTM C469	Cylinders 6x12	3	0.20	0.59
Modulus of Rupture	ASTM C78	Prism 6x6x20	3	0.42	1.25
Length Change	ASTM C157	Prism 3x3x11.25	3	0.06	0.18
Shrinkage Losses	NCHRP 496	Prism 4x4x24	4	0.22	0.89
				TOTAL	**4.32**

4.1 Slump-Flow Test – ASTM C1611

The slump-flow test is a workability test for fresh self-consolidating concrete. Much like the slump test for fresh normal concrete, it is a qualitative measure of the flowability and workability of the wet concrete. Equipment for the slump-flow test is a standard Abrams cone and flow table. The cone is used inverted, such that the smaller end is placed on the flow table, filled with fresh concrete and quickly raised to allow the concrete to flow across the table. Measurements of the maximum and minimum spread diameters are taken, and the average spread is calculated. There are no widely accepted rules that clearly define the requirements for an acceptable final spread diameter. In this project, and average spread in the range of 22 in. to 30 in. is considered acceptable. In addition, a visual stability index (VBI) of 0 or 1 is required. This index reflects the consistency and resistance to segregation of the mix. The higher the index, the lower the mix consistency and resistance to segregation. All the chosen NU UHPC mixes have shown adequate spread and high resistance to segregation, as shown in figures 4.1(a) and (b).

(a) (b)

Figure 4.1 (a) Slump-Flow Test and (b) Typical Aggregate Distribution

4.2 Compressive Strength Test – ASTM C39

Compressive strength of NU UHPC mixes was determined using 4 in. x 8 in. cylinders due to the capacity of the testing equipment (Forney; max 400,000 lb) and the high strength of the concrete. Cylinders were tested according to ASTM C39 at 1, 3, 7, 14, and 28 days after being stored in the moisture room until the testing day. Cylinder ends were ground using the Hi Kenma cylinder end grinder manufactured by Marui Co., LTD, as shown in figure 4.2, to ensure the consistency and reliability of test results.

Figure 4.2 Cylinder End Grinding and Testing

Three specimens were tested from each mix at a specific age and the average compressive strength was plotted versus time for each mix, as shown in figure 4.3. Detailed testing results are available in Appendix A. It should be noted that in some cases the average of only two cylinders was taken because a few cylinders did not comply with the ASTM C39 specifications and were eliminated.

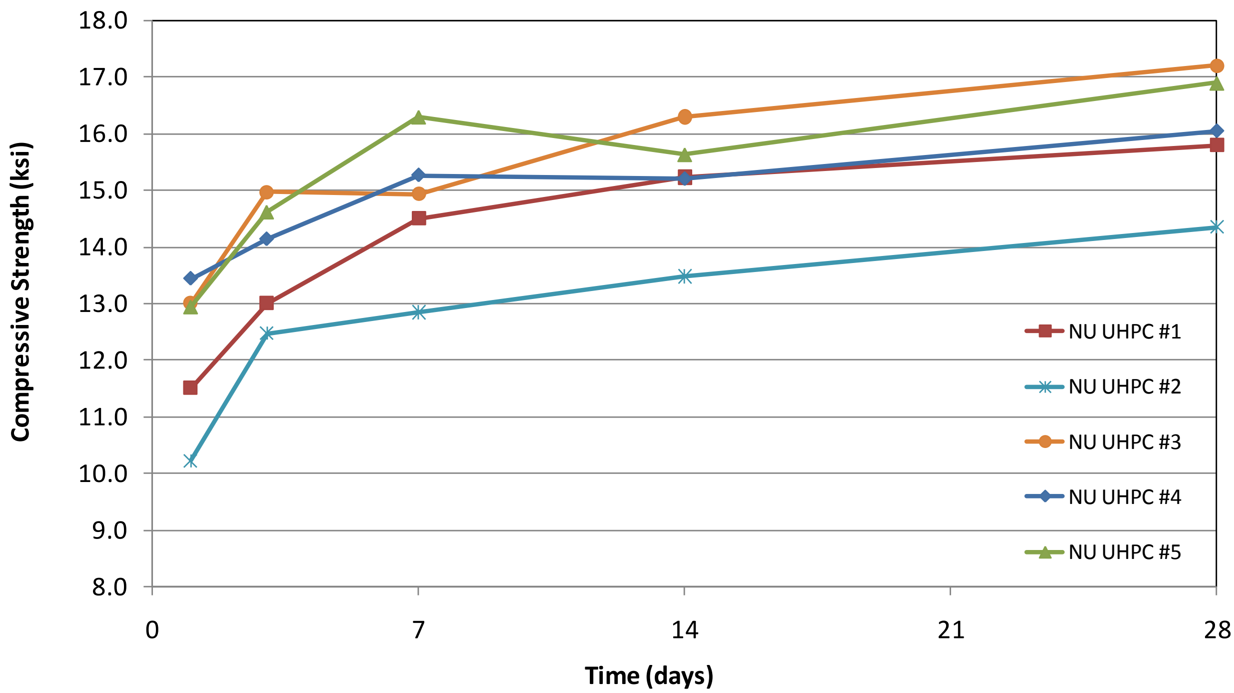


Figure 4.3 Average Compressive Strength versus Time for the Chosen NU UHPC Mixes

Since the concrete compressive strength is dependent on the type of curing used, the effect of two different curing procedures was evaluated. For NU UHPC mix #1, 15 cylinders were cured using standard moisture curing procedures according to ASTM C31, while another 15 cylinders were cured using accelerated curing procedures according to the PCI Architectural Quality Control Manual. The accelerated curing procedures were intended to emulate the effect of heat curing in a standard precast plant. These procedures are presented graphically in figure 4.4. Specimens are cast and immediately covered and left at the room temperature for 6 hours to allow for initial set. Specimens are then moved to an oven whose temperature is set at 90^{o}F. After one hour, the temperature is increased by 15^{o}F per hour for three hours until the oven temperature reaches 135^{o}F. The specimens are left at 135^{o}F for 9 hours. Then, the temperature is reduced in intervals of 10^{o}F per hour until the oven temperature reaches 90^{o}F. After one hour at 90^{o}F, the cylinder are removed from the oven, stripped, and placed in the moisture curing room.

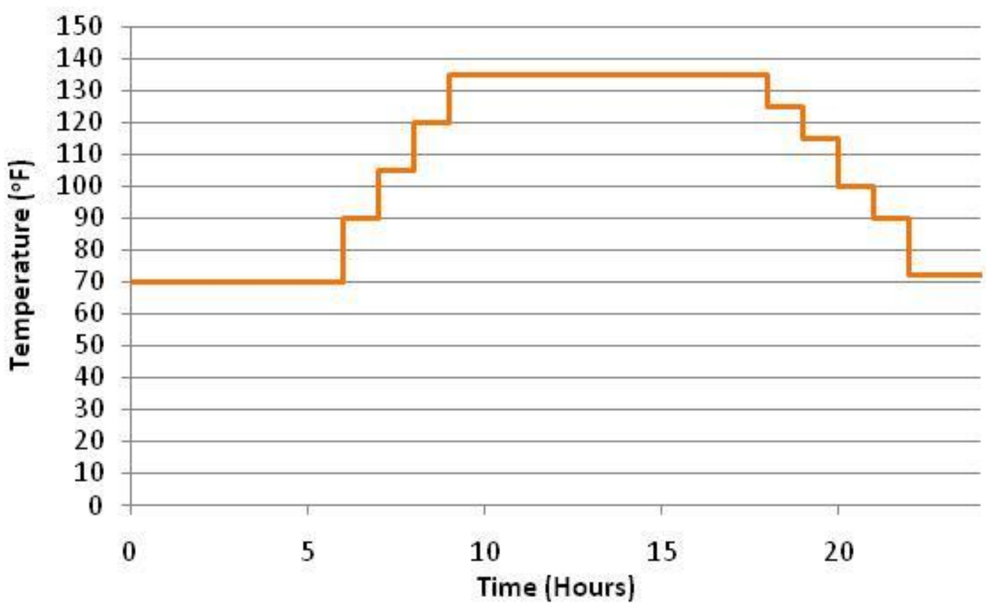


Figure 4.4 Temperature Setting Profile for Accelerated Curing

The cylinders of NU UHPC mix # 1 which were cured using the above-mentioned accelerated curing procedures and those that were moist cured were tested at 1, 3, 7, 14 and 28 days. The average compressive strength was plotted as shown in figure 4.5. This figure indicates that the accelerated curing procedure results in approximately 17% higher 1 day compressive strength and no increase in the final strength (28 days).

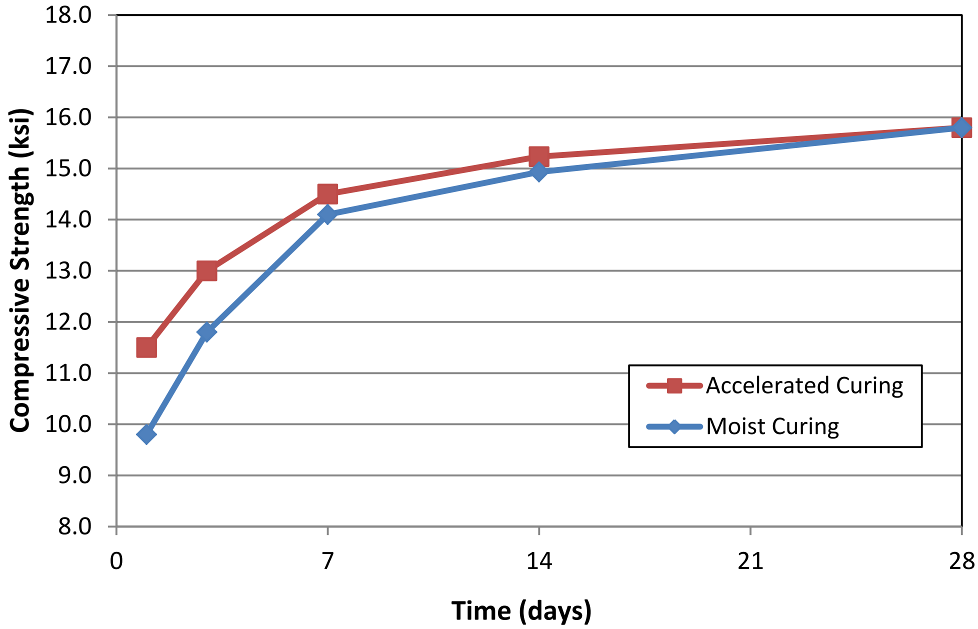


Figure 4.5 Average Compressive Strength versus Time - Accelerated and Moist Curing Conditions

For quality control purposes, the repeatability of the mixing procedures was evaluated by making NU UHPC mix #1 in two separate batches; A and B. For each batch, 15 cylinders were tested for compressive strength at 1, 3, 7, 15, and 28 days. Figure 4.6 shows the average compressive strength versus time of the two batches. The similarity of the two plots indicates adequate repeatability of mixing procedures.

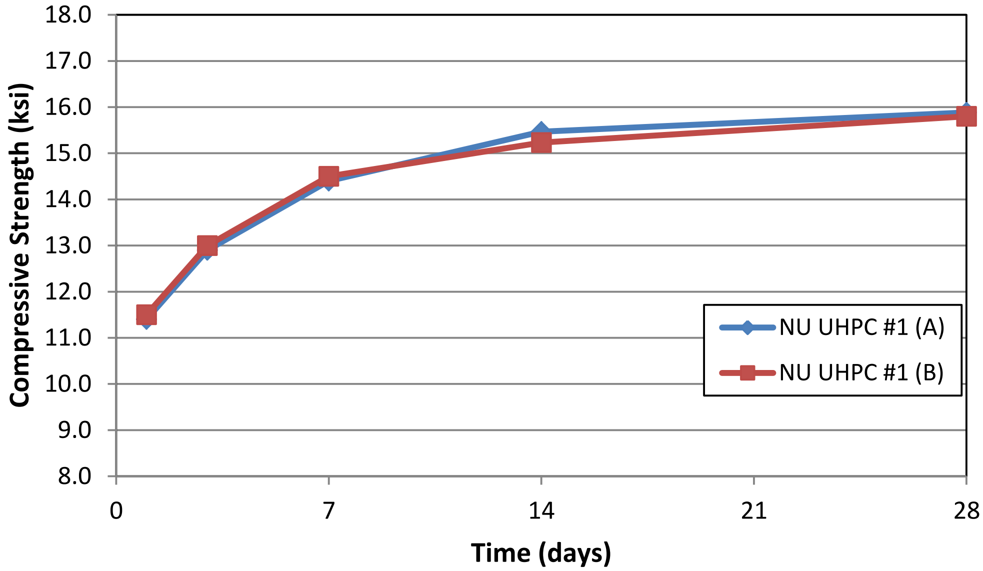


Figure 4.6 Average Compressive Strength versus Time - Evaluating Repeatability

4.3 Modulus of Elasticity Test – ASTM C469

Three 6 in. x 12 in. cylinders from each NU UHPC mix were tested for modulus of elasticity (MOE) according to ASTM C469 at 28 days. A sulfur-based capping compound was used for capping each cylinder and a combined compressometer and extensometer was attached, as shown in figure 4.7. Each cylinder was loaded four times to approximately 40% of its compressive strength, with the first loading performed solely to seat the gauges and the subsequent three loadings were used to determine an average MOE, which is plotted for each NU UHPC mix in figure 4.8. Detailed testing results are available in Appendix A.

Figure 4.7 Modulus of Elasticity Test Setup

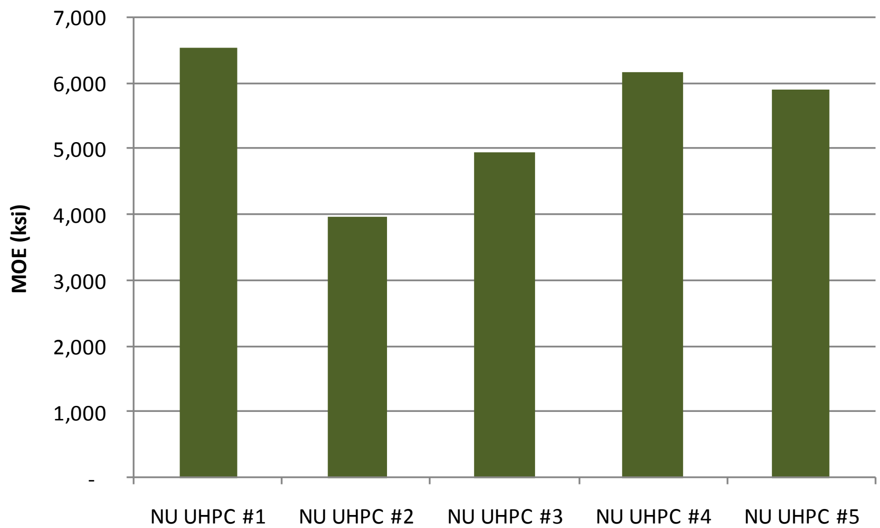


Figure 4.8 Average Modulus of Elasticity for the Chosen NU UHPC Mixes

4.4 Splitting Strength Test – ASTM C496

The splitting strength test was performed according to ASTM C496 at 28 days to determine the split tensile capacity of each mix. Three 6 in. x 12 in. cylinders were loaded to failure, as shown in figure 4.9, using the Tinius Olson Testing Machine to determine the average splitting strength. The average splitting tensile strength of each NU UHPC mix is plotted as shown in figure 4.10. Detailed testing results are available in Appendix A.

Figure 4.9 Splitting Strength Test Setup

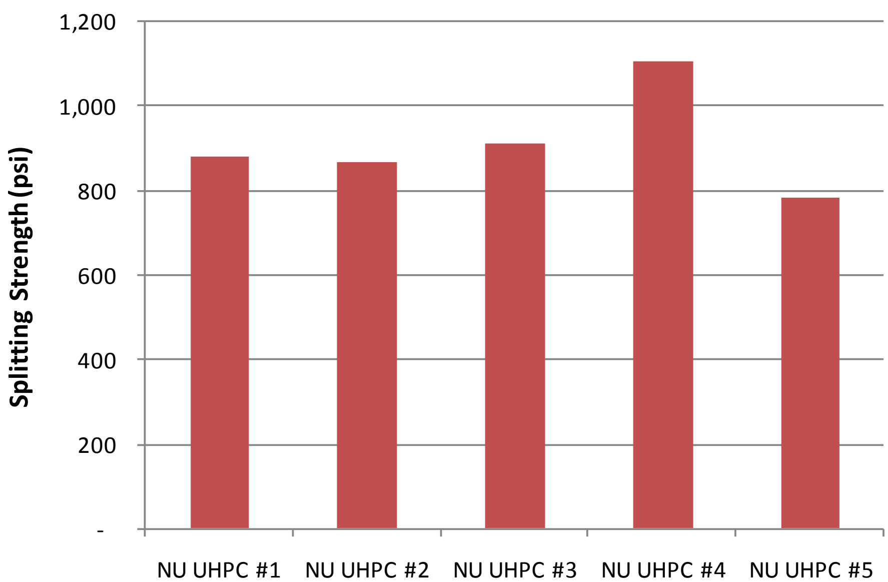


Figure 4.10 Average Splitting Strength for the Chosen NU UHPC Mixes

4.5 Flexure Strength Test – ASTM C78

Three 6 in. x 6 in. x 20 in. prisms from each mix were tested for flexure strength according to ASTM C78 using third-point loading, as shown in figure 4.11. The Tinius Olson Testing Machine was used to load the specimens to failure. The width and depth of each prism was measured to accurately calculate the modulus of rupture (MOR). All beams fractured inside the middle third of the span (+/- 5%); therefore, all results are valid. The average rupture strength of each NU UHPC mix is plotted in figure 4.12. Detailed testing results are available in Appendix A.

Figure 4.11 Flexure Strength Test Setup

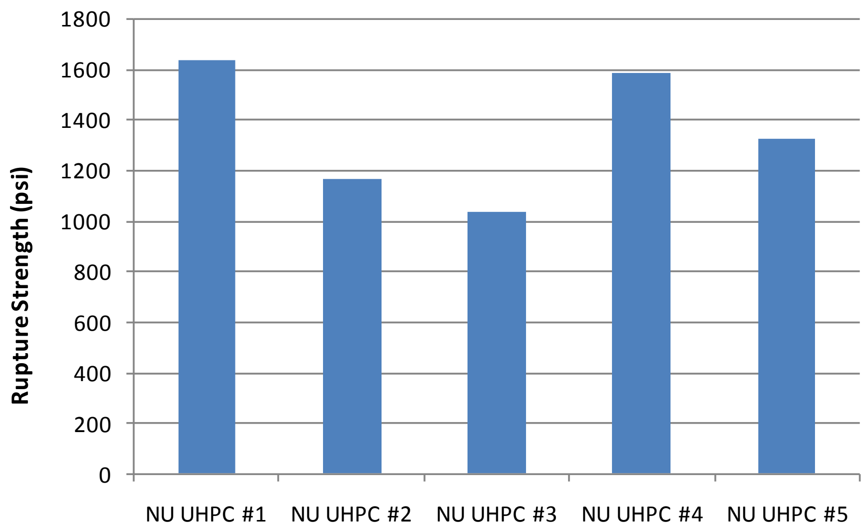


Figure 4.12 Average Flexure Strength for the Chosen NU UHPC Mixes

4.6 Length Change Test – ASTM C157

Three 3 in. x 3 in. x 11 ¼ in. specimens from each NU UHPC mix were tested for length change according to ASTM C157. At an age of 23 ½ hours (+/- ½), each specimen was removed from its mold, placed in lime-saturated water for 30 minutes and then placed in the comparator for its initial reading, shown in figure 4.13. Then the specimen was returned to the lime-saturated water for 27 days. Length measurement of the specimen and reference were taken at different ages. From these measurements, the change in length of each sample was calculated. Table 4.3 lists the 28-day length change percentages for the five chosen NU UHPC mixes. These results indicate that these mixes may exhibit shrinkage (negative length change) or expansion (positive length change) during the first 28 days in wet conditions. Detailed testing results are available in Appendix A.

Figure 4.13 Comparator with Reference Bar and Specimen

Table 4.3 Average Length Change at 28 days for the Chosen NU UHPC Mixes

Mix	Length Change at 28 days
NU UHPC #1	0.12%
NU UHPC #2	-0.06%
NU UHPC #3	-0.04%
NU UHPC #4	0.09%
NU UHPC #5	-0.03%

4.7 Final Mixes

Based on the results of material tests performed in the chapter, mix #4 and mix #5 were selected as “Final Mixes”. These two mixes represent the best two mixes without and with coarse aggregate, respectively, in terms of flowability and mechanical properties. Therefore, the two mixes were further refined for additional testing and designated NU UHPC mix #4’ and mix #5’ respectively. The application of these two mixes to the design and production of bridge I-girders will be presented in the next chapter. All material tests presented earlier, in addition to the shrinkage losses test, are performed on these two mixes to confirm their mechanical properties required for the design and production of precast prestressed bridge girders. Table 4.4 lists the design and material cost of the final mixes.

Table 4.4 Design and Cost of Final NU UHPC Mixes

Material Type	NU UHPC #4'		NU UHPC #5'	
	Weight in Pounds per	**Material Cost ($)**	**Weight in Pounds per**	**Material Cost ($)**
#10 Sand	2255	$ 11.3	1580	$ 7.9
1/4" BRS	0	$ -	672.3	$ 5.0
Cement III	1120	$ 50.4	1050	$ 47.3
C Fly Ash	240	$ 1.8	300	$ 2.3
Silica Fume	240	$ 72.0	150	$ 45.0
Chryso	70.0	$ 78.7	54.0	$ 60.7
Agg. Water	33.0	$ -	33.0	$ -
Free Water	232.0	$ -	212.0	$ -
W/C ratio	**0.166**	**$ 214.1**	**0.163**	**$ 168.1**

Three 4 in. x 8 in. cylinders from each mix were tested for compressive strength at ages 1, 3, 7, 14, 28, 56, and 105 days. Figure 4.14 plots the compressive strength versus age relationships that best fits the data points for the two final mixes. This figure clearly indicates the consistency of test results and the steady gain of compressive strength with time. Test results also indicate that the compressive strength of the two mixes exceeded 12 ksi at 24 hours, 15 ksi at 28 days, and 16 ksi at 56 days. It should be noted that all cylinders were end ground and had one day of accelerated curing followed by moist curing until the time of testing.

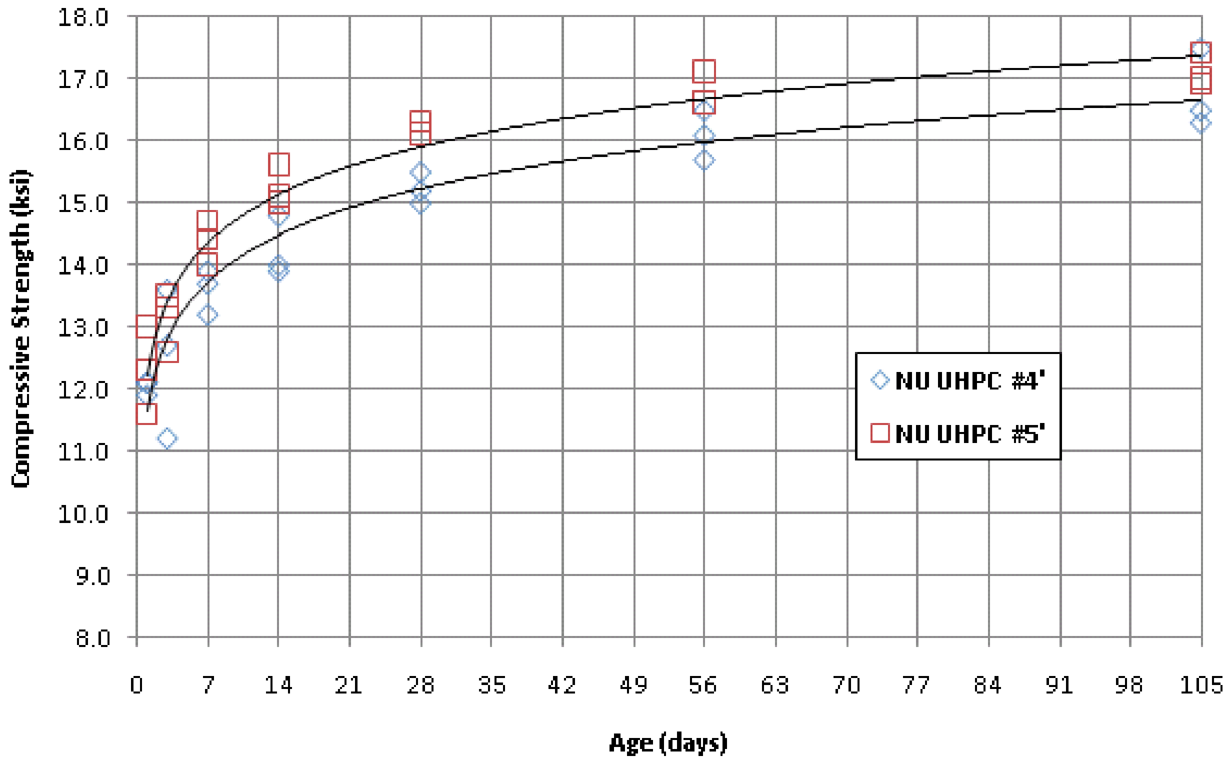


Figure 4.14 Compressive strength versus Age for final NU UHPC mixes

Table 4.5 shows the results of modulus of elasticity (MOE) testing at 28 days and the values calculated using the 2007 AASHTO LRFD equation 5.4.2.4-1 ($E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_c}$). The calculated MOE values are based on the average compressive strength at 28 days, K_1 = 1.0, and unit weight of 148 lbs/ft^3, and 149 lbs/ft^3 for mixes #4' and #5' respectively. These unit weight values were obtained using ASTM C136 test method. Table 4.5 indicates that the MOE values calculated using AASHTO LRFD specifications are approximately 19% higher than the measured values. This is consistent with the findings of other research programs on HPC and UHPC concretes (Mokhtarzadeh and French, 2000; Ma and Schneider 2002). It should be also noted that the K_1 factor of the AASHTO LRFD equation is used to account for the effect of aggregate type on the MOR of the concrete. Using a K_1 value of 0.85 for NU UHPC mixes will result in MOE values very close to the measured ones. Further testing is required to obtain more accurate estimate of the K_1 factor for the aggregate used.

Table 4.5 Modulus of Elasticity of Final NU UHPC Mixes (ksi)

MOE	NU UHPC #4'				NU UHPC #5'			
Specimen	1	2	3	Average	1	2	3	Average
#1	5,968	6,015	6,017	**6,000**	6,517	6,429	6,345	**6,430**
#2	6,168	6,169	6,121	**6,153**	6,645	6,607	6,564	**6,605**
#3	6,195	6,238	6,250	**6,228**	6,388	6,257	6,339	**6,328**
			Measured	**6,127**			**Measured**	**6,454**
			Calculated	**7,333**			**Calculated**	**7,631**

Table 4.6 shows the results of splitting tensile strength testing at 28 days and the values calculated using the 2007 AASHTO LRFD equation in C5.4.2.7 ($f_t = 0.23\sqrt{f_c^{'}}$).Splitting stress calculations are based on the average compressive strengths at 28 days for mixes #4' and #5'. Table 4.6 indicates that the calculated values are within ± 8% of the measured values.

Table 4.6 Splitting Tensile Strength of Final NU UHPC Mixes (ksi)

Splitting	NU UHPC #4'				NU UHPC #5'			
Specimen	Diameter (in)	Length (in)	Load (lbs)	Strength (ksi)	Diameter (in)	Length (in)	Load (lbs)	Strength (ksi)
#1	5.97	12.07	129,400	**1.14**	5.97	12.00	89,200	**0.79**
#2	6.02	12.07	87,800	**0.77**	5.98	12.04	96,100	**0.85**
#3	5.99	12.07	96,800	**0.85**	5.99	12.08	102,500	**0.90**
			Measured	**0.92**			**Measured**	**0.85**
			Calculated	**0.90**			**Calculated**	**0.92**

Table 4.7 shows the results of flexure testing at 28 days and the values of the modulus of rupture (MOR) calculated using the 2007 AASHTO LRFD equations in C5.4.2.6 ($f_r = 0.24\sqrt{f_c'}$ to $0.37\sqrt{f_c'}$). Calculations for MOR used the average compressive strengths at 28 days for mixes #4' and #5'. Table 4.7 shows that the measured MOR is within the calculated range for NU UHPC mix #5 and higher than the calculated upper limit for NU UHPC mix # 4, which indicates the high resistance to cracking of the developed mixes.

Table 4.7 Flexure Strength of Final NU UHPC Mixes (ksi)

MOR	**NU UHPC #4'**				**NU UHPC #5'**			
Specimen	**Width (in)**	**Height (in)**	**Load (lbs)**	**Strength (ksi)**	**Width (in)**	**Height (in)**	**Load (lbs)**	**Strength (ksi)**
#1	5.85	6.01	20,100	**1.71**	6.12	6.07	17,400	**1.39**
#2	6.04	6.02	19,720	**1.62**	5.96	6.08	16,170	**1.32**
#3	5.93	6.03	18,960	**1.58**	6.00	6.97	16,450	**1.02**
			Measured	**1.64**			**Measured**	**1.24**
			$0.24\sqrt{f_c'}$	**0.94**			$0.24\sqrt{f_c'}$	**0.96**
			$0.37\sqrt{f_c'}$	**1.44**			$0.37\sqrt{f_c'}$	**1.49**

Table 4.8 shows the measured change in length of water cured NU UHPC mixes 5' and 4' at 28 days. These results indicate that both mixes experienced small amounts of shrinkage over the 28 day period. It should be noted that the NU UHPC mix 5' has a smaller shrinkage (negative length change), which was expected due to the restraining effect of the aggregates.

Table 4.8 Length Change of Final NU UHPC Mixes

Length Change	NU UHPC #4'	NU UHPC #5'
Specimen	**28-days**	**28-days**
#1	-0.030%	-0.041%
#2	-0.038%	-0.020%
#3	-0.034%	-0.032%
Average	**-0.034%**	**-0.031%**

The NCHRP Report 496 (Tadros et al. 2003) presents a method for measuring the shrinkage of concrete for prestress loss calculations. This method, which was developed by researchers at the University of Nebraska-Lincoln, resulted in the current AASHTO LRFD equation for calculating concrete shrinkage. The same method was used to measure the shrinkage of the two final NU UHPC mixes. Four concrete specimens 4 in. x 4 in. x 24 in. were cast from each mix in steel molds, as shown in figure 4.15(a). An extra specimen was cast from each mix in case a specimen was not usable. Detachable Mechanical Strain Gauge (DEMEC) points were attached to two opposing sides lengthwise of each specimen as shown in figure 4.15(b). Five DEMEC points were attached to each side at 4 in. spacing, which results in three readings each side (readings are taken every other point). Readings were taken using a dial gauge reader, manufactured by W.H. Mayes & Son. The specimens were then allowed to cure 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, and once per month thereafter. Figures 4.16 and 4.17 plot the measured shrinkage strains versus time for NU UHPC Mix 4’ and 5’, respectively.

(a) (b)

Figure 4.15 Shrinkage Specimens of Final NU UHPC Mixes

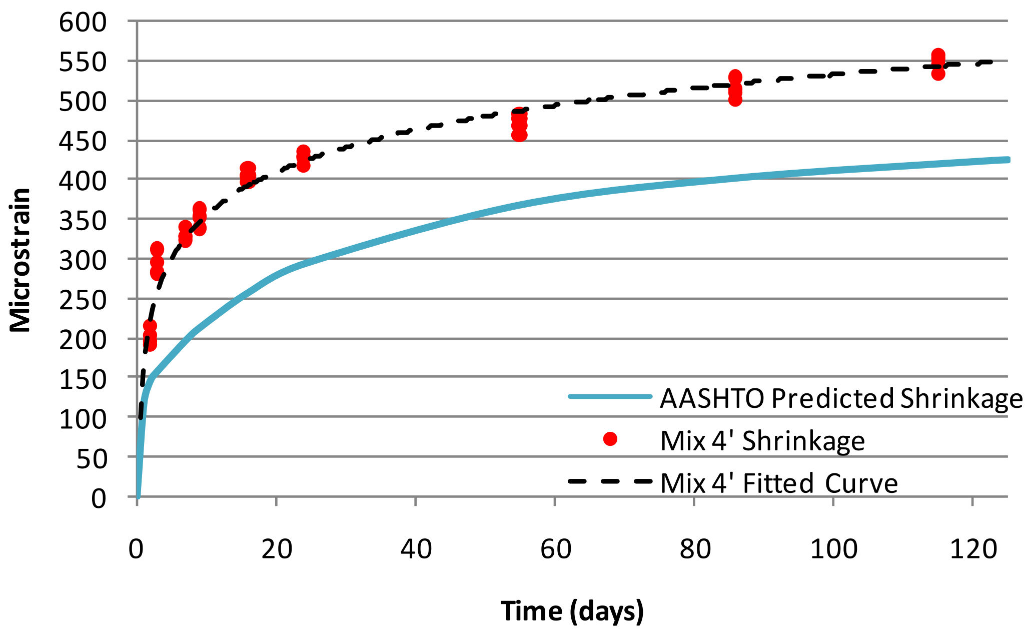


Figure 4.16 Shrinkage Strain versus Time for NU UHPC Mix #4’

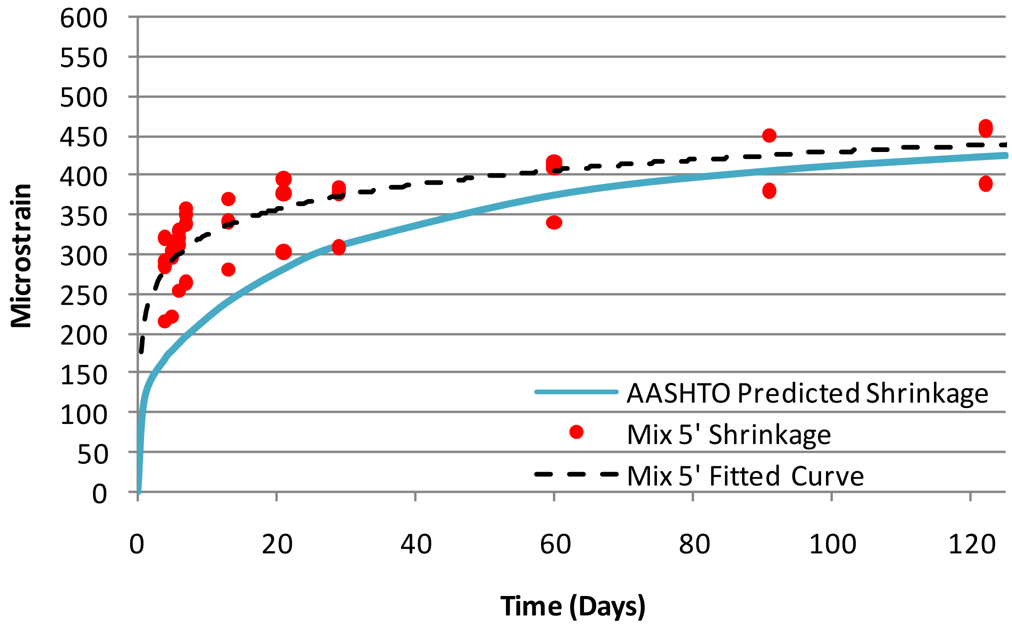


Figure 4.17 Shrinkage Strain versus Time for NU UHPC Mix #5'

Figures 4.16 and 4.17 also plot the shrinkage strain calculated using the 2007 AASHTO LRFD equations of Section 5.4.2.3. Comparing the measured shrinkage strains against the AASHTO predicted strains indicate that the current AASHTO method provides a good estimate of the long-term shrinkage strain for NU UHPC mix #5', while it significantly underestimates the strain for NU UHPC mix #4'. This is because mix # 4' has no coarse aggregate and it contains a larger amount of cementitious materials that results in higher shrinkage strains. Also, it should be noted that the current AASHTO equations are applicable to concrete strengths of 15 ksi or less, while the strength of the developed mixes exceeds this limit.

Chapter 5 Applications of NU UHPC to Bridge I-Girders

This chapter presents two applications of the final NU UHPC mixes developed within this project. The first application is the use of NU UHPC mix #4' in the production of two AASHTO type II girders. These girders are tested to evaluate the shear capacity of UHPC reinforced with welded wire reinforcement (WWR) instead of random steel fibers. These tests were performed as part of another research project sponsored by the Welded Wire Institute (WRI). The second application is the use of NU UHPC mix #5' in the production of a NU900 girder prestressed using 30 – 0.7 in. strands. This girder was tested to evaluate the development length of 0.7 in. strands as part of another research project sponsored by Nebraska Department of Roads (NDOR) for investigating the impact of using 0.7 in. strands at 2 in. spacing.

5.1 Application # 1: AASHTO Type II Girders

In this application, two AASHTO Type II girders were fabricated at Coreslab Structures Inc.– Omaha using NU UHPC mix 4'. The girders were 18 ft- 6 in. long and have the cross-section shown in figure 5.1. The girders were pretensioned using 24–0.6 in. grade 270 low-relaxation prestressing strands tensioned to $0.75f_{pu}$. Mild steel, used as compression reinforcement, was 2#6 and 2#9 grade 60 bars. Two partially prestressed (f_{pj} = 102 ksi) 0.6 in. strands were used to control cracks at release. Shear reinforcement consisted of two grade 80 – 4 in. x 4 in. – D16 x D16 WWR meshes. The end zone was reinforced using four ¾ in. headed coil rods at 2 in. spacing along the girder axis welded to the bearing plate. The bottom and top flanges were reinforced using D11 WWR at 6 in. spacing along the girder length for confinement, as shown in figure 5.2. No deck was placed on either of the AASHTO Type II girders to compare the performance of these girders against the performance of the girders tested by the FHWA in 2001, which were fabricated using a commercial UHPC and reinforced with random steel fibers. The main objective of this comparison was to evaluate the structural capacity and economy of using WWR versus random steel fibers in UHPC girders.

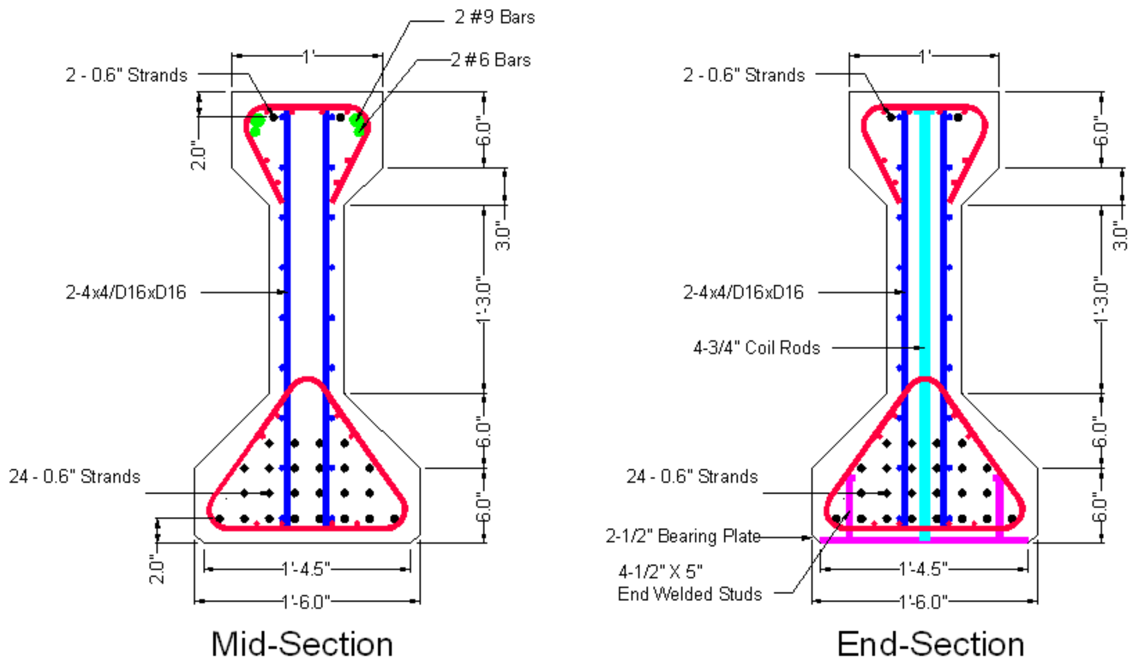


Figure 5.1 Cross Section of AASHTO Type II girders

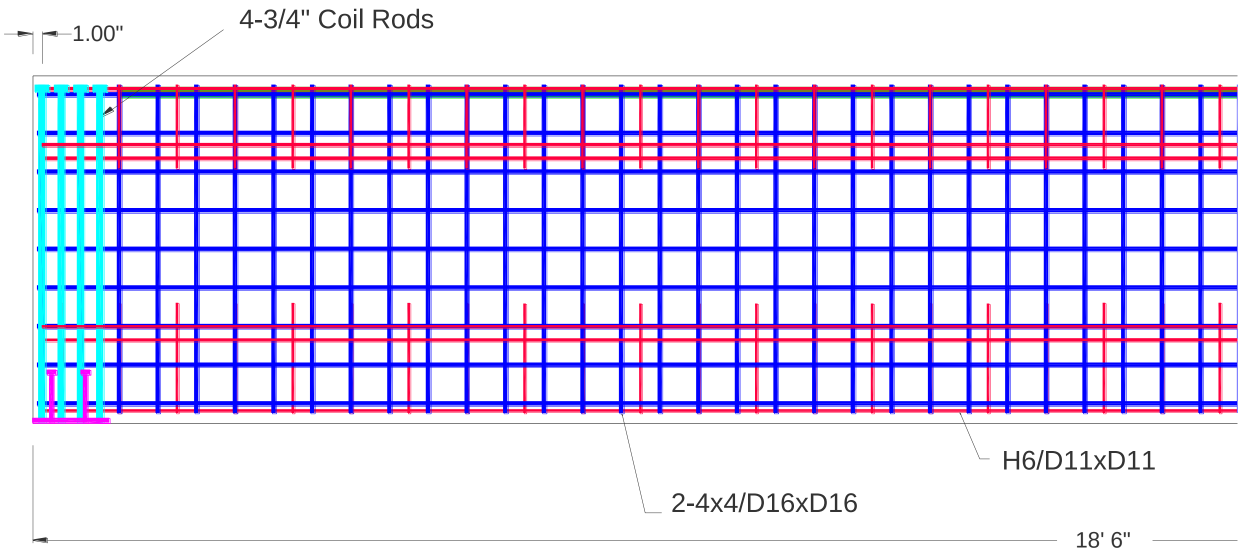


Figure 5.2 Elevation of AASHTO Type II Girders

The two AASHTO Type II girders were designed using a specified release strength of 8 ksi and a final strength of 15 ksi. The NU UHPC mix #4' was applied using two different mixing procedures. The first mixing procedure was applied to AASHTO Type II girder A as follows:

1. Mix cementitious materials with all water and HRWR for 2-3 minutes;
2. Add fine sand and mix for 10 - 15 minutes;
3. Transport the concrete using a truck mixer and check slump-flow on-site;
4. Add HRWR if needed (average spread diameter is less than 22 in.)

This procedure did not work well as the concrete was very lumpy and had poor flowability. Large amounts of HRWR were added in order to achieve the minimum required spread, as shown in figure 5.3. Additionally, this procedure produced excessive heat that affected the strength gained over time.

Figure 5.3 Flowability of NU UHPC Mix #4' Used for AASHTO Type II Girder A

The second mixing procedure was applied to AASHTO Type II girder B as follows:

1. All granular materials are pre-blended for 2-3 minutes (dry mixing);
2. All water and HRWR are added;
3. Mixing continues until adequate flowability is achieved (10 to 15 minutes);
4. Transport the concrete using a truck mixer and check slump-flow on-site;
5. Add HRWR if needed (average spread less than 22 in.)

This procedure was very successful and resulted in a very flowable concrete with an average spread of 30 in., as shown in figure 5.4.

Figure 5.4 Flowability of NU UHPC Mix #4’ Used for AASHTO Type II Girder B

Cylinders were taken from the two batches and tested for compressive strength at different ages. For AASHTO Type II girder A, the unsuccessful mixing procedure resulted in a high heat of hydration that caused the development of micro-cracks over time and negatively affected the compressive strength, as shown in figure 5.5. Although a compressive strength of 18 ksi was achieved after 3 days, the strength continued to decline with time to less than 15 ksi after 56 days.

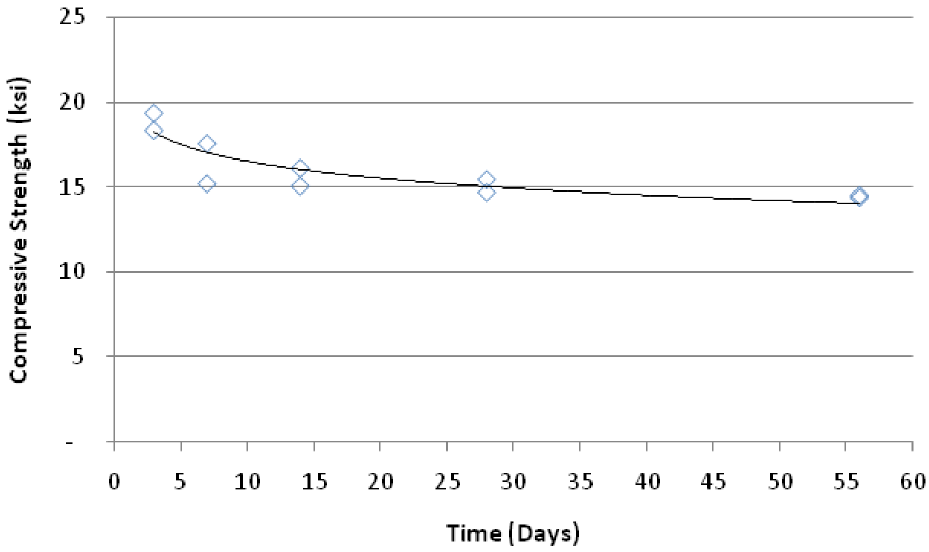


Figure 5.5 Compressive Strength versus Time for AASHTO Type II Girder A

For the AASHTO Type II girder B, the successful mixing procedure resulted in a 1-day compressive strength of 13 ksi and continued to rise until it reached 21 ksi at the time of testing (130 days), as shown in figure 5.6

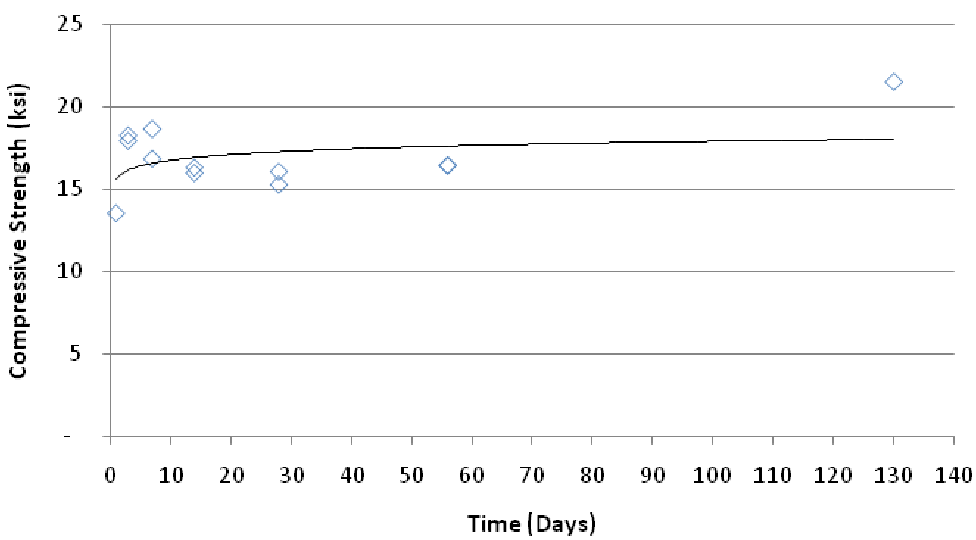


Figure 5.6 Compressive Strength versus Time for AASHTO Type II Girder B

The two girders were tested by applying a third point loading, which was 6 ft from the centerline of bearing, as shown in figure 5.7. Vertical deflections were measured using a string potentiometer directly under the loading point. Strain gauges were attached to the top and bottom flanges and the web of the girders to measure the strain profiles at different loading stages. Figure 5.8 shows the load-deflection relationship of the two test girders.

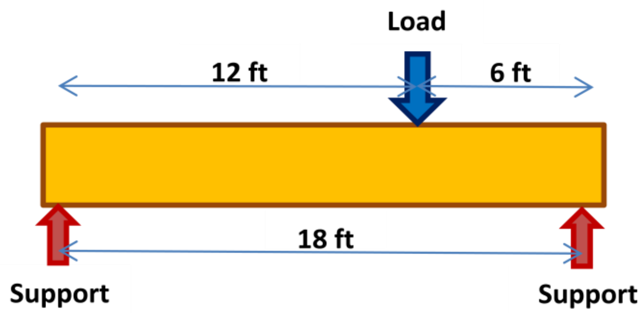


Figure 5.7 Test Setup for AASHTO Type II Girders

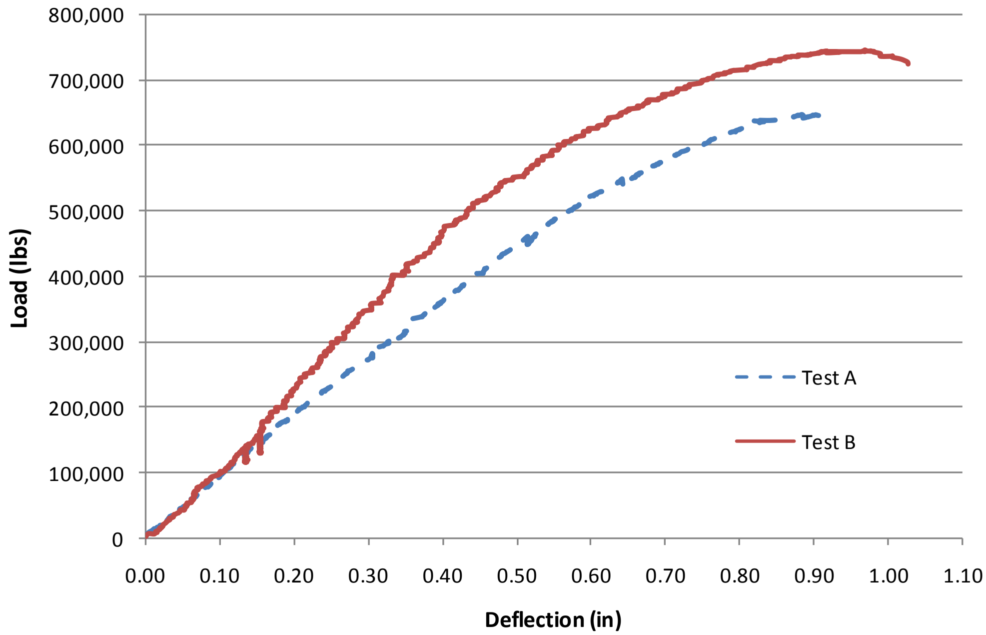


Figure 5.8 Load–Deflection Relationship for Test Girders A and B

Figure 5.8 indicates that the two girders behaved linearly up to the flexure cracking load, which was predicted to be 419 kips. Afterwards, the load-deflection relationship changes from linear to non-linear up to the ultimate load, which was 648 kip for girder A and 746 kip for girder B. Table 5.1 lists the predicted and observed cracking and ultimate loads for both flexure and shear. Loads were predicted using the 2007 AASHTO LRFD specifications.

Table 5.1 Predicted Loads and Applied Loads

Point Load Applied (kips)	Flexural	Shear
Predicted Cracking Load*	419	118
Observed Cracking Load A	425	150
Observed Cracking Load B	450	150
Predicted Maximum Capacity*	1194	642
Maximum Applied Load A	648	
Maximum Applied Load B	746	

* Predicted using specified material properties and accounting for the underdeveloped strands

Girders A and B, after failure, can be found in figures 5.9 and 5.10, respectively. Both girders exhibited heavy spalling of the web concrete and large cracks at the diaphragm concrete. The failure mode was identified to be shear/bond failure as indicated by the diagonal shear cracks at the web between the support and loading point. The shear failure was initiated by the bond failure of prestressing strands due to inadequate development length and insufficient anchorage in the diaphragm concrete.

Figure 5.9 Failure of Girder A

Figure 5.10 Failure of Girder B

Based on the testing results, a comparison between the two prestressed AASHTO Type II girders made of NU UHPC mix #4' with WWR and the three AASHTO Type II girders made of commercial UHPC with random steel fibers was made, as shown in table 5.2. The comparison includes the average shear capacity and material cost of the two sets of girders. Based on this comparison, it can be easily concluded that the NU UHPC mix 4' reinforced with WWR outperformed the commercial UHPC with steel fibers while being 65% more economical.

Table 5.2 Shear Capacity and Material Cost Comparison of Commercial UHPC and NU UHPC

Material (Estimated Cost)	Span Length (ft)	Shear Span (ft)	Applied Load (kips)	Applied Shear (kips)	Average Shear Capacity (kips)
UHPC with Steel Fibers **($1000/yd^3)**	28	6.5	500	384	**442**
	24	7.5	731	503	
	14	6	766	438	
NU UHPC with WWR **($350/yd^3)**	18	6	648	432	**465**
	18	6	746	498	

5.2 Application # 2: NU900 Girder

In this application, a 40 ft long NU900 girder was fabricated at Coreslab Structures Inc.–Omaha using NU UHPC mix #5'. The girder cross-section can be found in figure 5.11. The girder was pretensioned using 30–0.7 in. grade 270 low-relaxation prestressing strands tensioned to 0.66f_{pu} due to the limited bed capacity. Mild steel was used for top flange reinforcement and deck reinforcement, as shown in figure 5.11. The girder was heavily reinforced in shear using two 6 x 6 – D31 x D31 WWR meshes to ensure that the girder will fail in flexure and not in shear when the test load. The end zone was reinforced using four #6 bars at 2 in. spacing along the girder axis in addition to studded bearing plates. The bottom flange was reinforced using D11 WWR at 6 in. spacing along the girder length for confinement, as shown in figure 5.12. After release, an 8.5 in. thick cast-in-place deck with a final concrete strength of 12 ksi was placed over the top flange (4 ft wide) to simulate a 12 ft wide, 4 ksi concrete deck in real bridge applications. In addition, two half-depth cast-in-place diaphragms with 6 ksi concrete strength were poured at girder ends to anchor ten bent strands, as shown in figure 5.13, which simulates the NDOR current practice in bridge construction using NU girders.

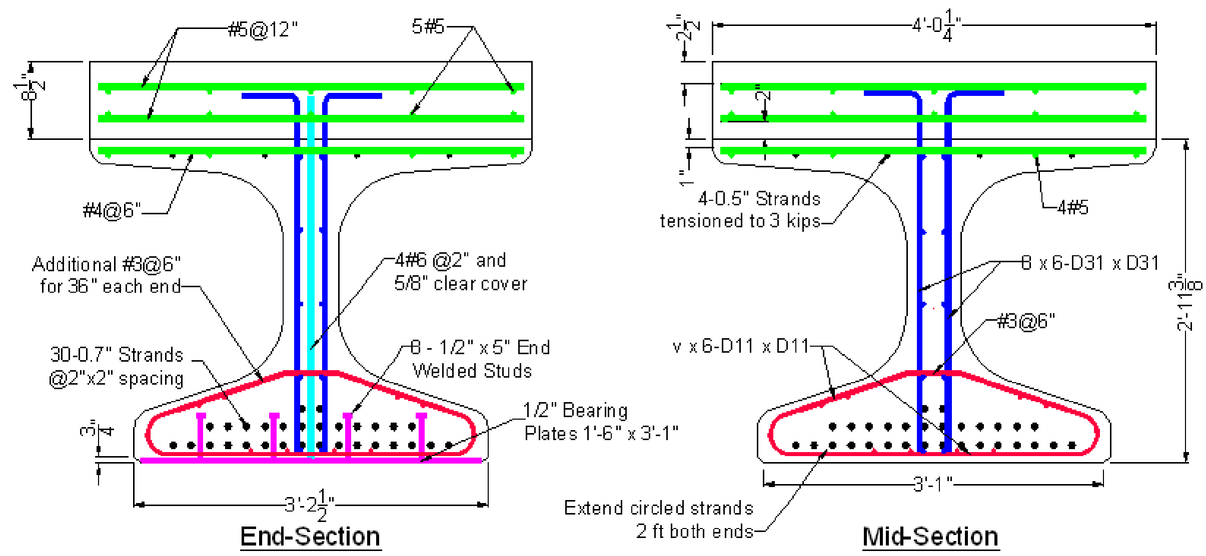


Figure 5.11 End and Mid Cross Sections of the NU900 Girder

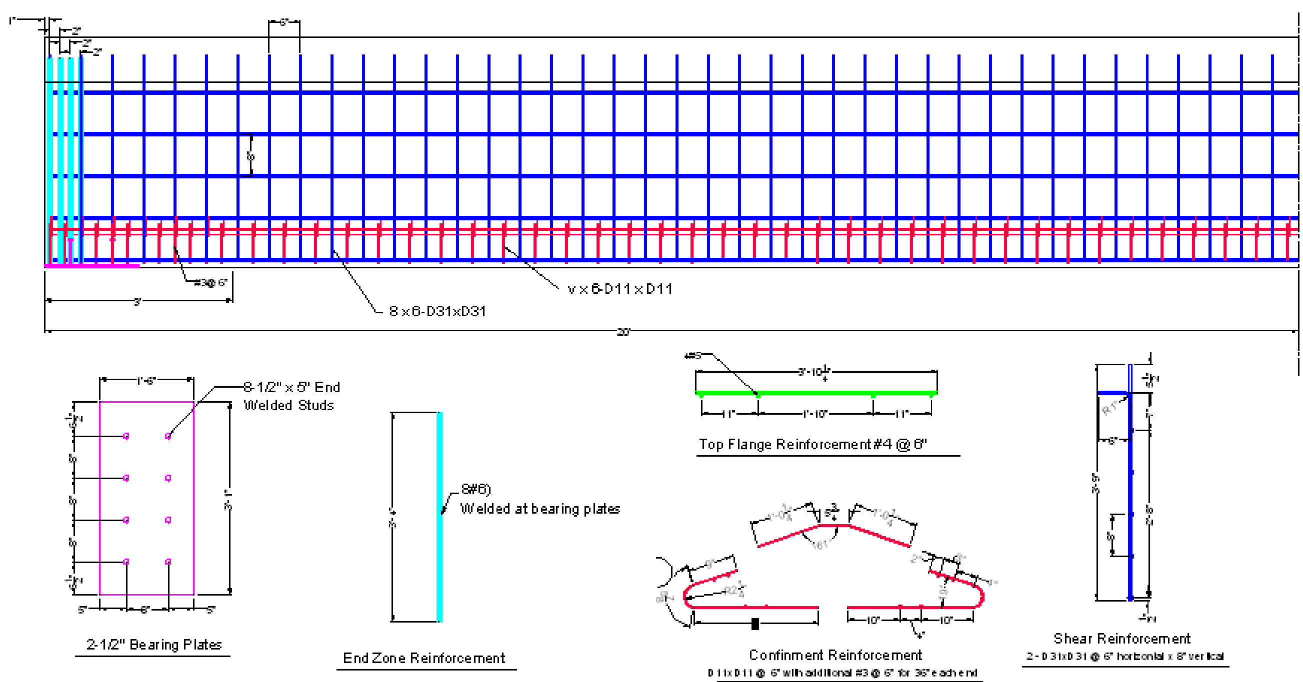


Figure 5.12 Elevation and Reinforcement Detail of NU900 Girder

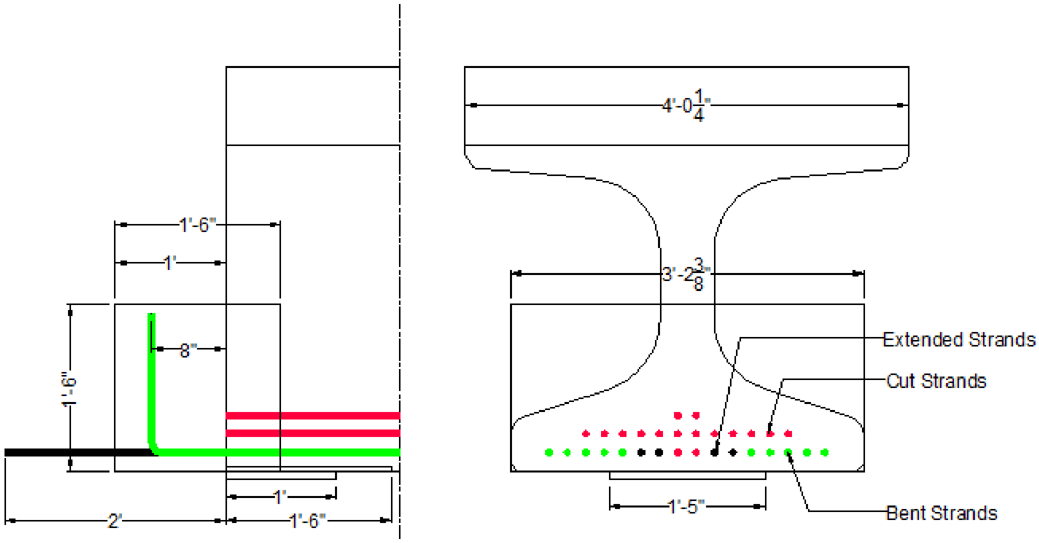


Figure 5.13 Bent, Cut, and Extended Strands at Girder Ends

The NU900 girder made of NU UHPC mix #5' was specified with a minimum concrete strength of 10 ksi at release and 15 ksi at 28 days. The mixing procedure used in this application was similar to the procedure followed in AASHTO Type II girder B application. The procedure was very successful and resulted in a very flowable concrete with an average spread of 30 in., as shown in figure 5.14.

Figure 5.14 Flowability of NU UHPC Mix #5’ Used in NU900 Girder

Figure 5.15 shows the compressive strength of the NU UHPC Mix #5’ used in NU900 girder production versus time. The average compressive strength based on three testing cylinders was over 12 ksi at release (1 day), 15.5 ksi at final (28 days), over 17 ksi at the time of testing (78 days). Table 5.3 lists the compressive strength test results for the deck and diaphragm cast-in-place concrete at the time of testing. This table indicates that the deck and diaphragm concrete had a much higher compressive strength than specified. Table 5.4 lists the other material properties of the NU UHPC mix #5’ used in NU900 girder.

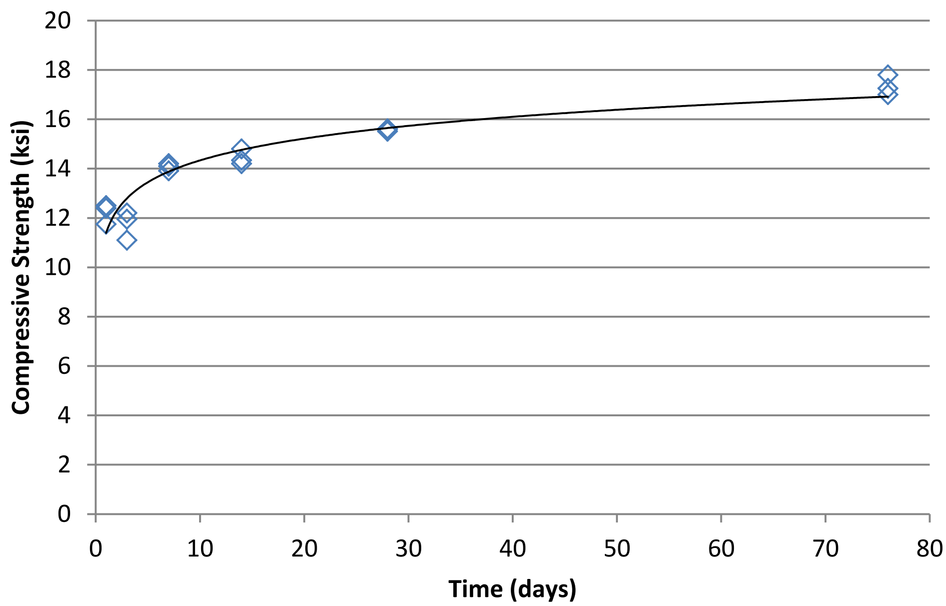


Figure 5.15 Compressive Strength of NU UHPC Mix #5' Used in NU900 Girder

Table 5.3 Compressive Strength of the Deck and Diaphragm Concrete

Specimen	Deck	Diaphragm
#1	13,741	8,944
#2	15,203	8,745
#3	14,326	9,864
Average Compressive Strength (psi)	**14,423**	**9,184**
Specified Compressive Strength (psi)	**12,000**	**6,000**

Table 5.4 Material Properties of NU UHPC Mix #5' Used in NU900 Girder

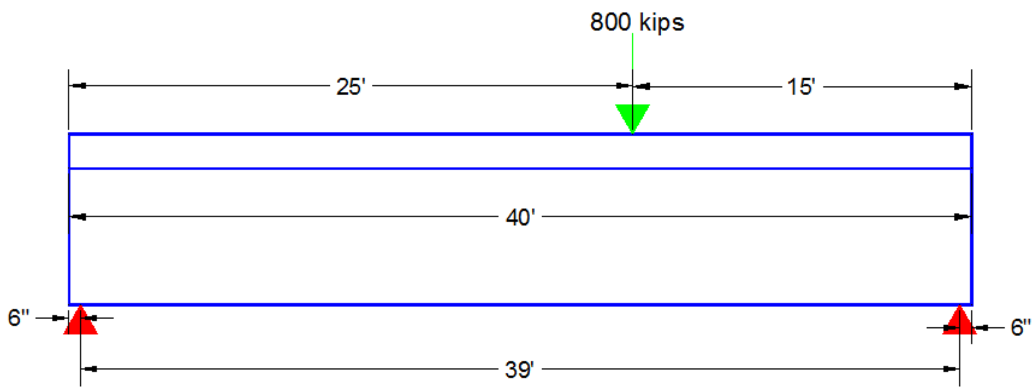
Specimen	#1	#2	#3	Average
Average Depth (in)	6.065	6.079	6.966	**6.37**
Average Width (in)	6.119	5.963	6	**6.03**
Load at Failure (lb)	17,400	16,170	16450	**16,673**
Modulus of Rupture (psi)	**1,391**	**1,321**	**1,017**	**1,243**

Specimen	#1	#2	#3	Average
Diameter (in)	5.99	6.022	5.993	**6.00**
Length (in)	11.95	11.94	11.89	**11.93**
Load (lb)	105,500	86,800	124,600	**105,633**
Splitting Tensile Strength (psi)	**938**	**769**	**1,113**	**940**

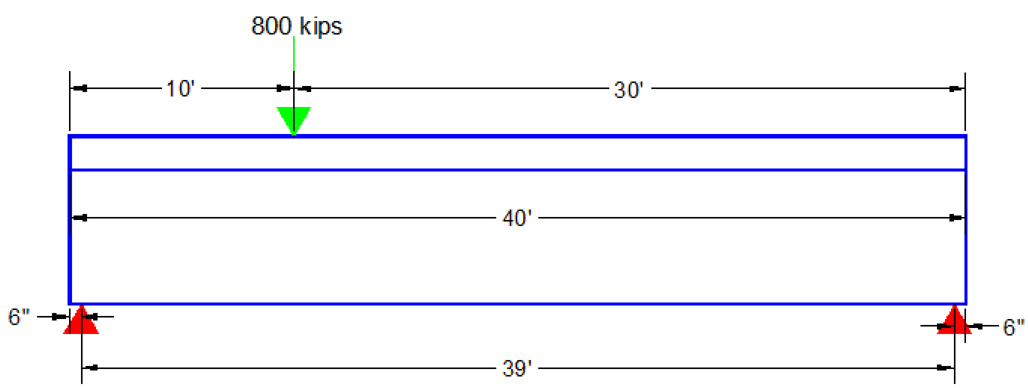
Specimen	#1	#2	#3	Average
Diameter (in)	5.93	5.99	6.01	**5.98**
Length (in)	11.92	11.98	11.95	**11.95**
Modulus 1 (ksi)	5,920	5,840	6,150	**5,970**
Modulus 2 (ksi)	5,960	5,940	6,230	**6,043**
Modulus 3 (ksi)	6,170	5,840	6,200	**6,070**
Modulus of Elasticity (ksi)	**6,017**	**5,873**	**6,193**	**6,028**

Time (days)	Datum	# 1	# 2	Average	Difference	Length Change
1	0.3612	0.3774	0.4066	0.392	-0.031	0.000%
3	0.3643	0.3784	0.4098	0.394	-0.030	0.010%
7	0.3689	0.3823	0.4109	0.397	-0.028	0.031%
14	0.3709	0.3907	0.4112	0.401	-0.030	0.007%
28	0.353	0.3771	0.4058	0.391	-0.038	-0.076%

The NU900 girder was first tested, as shown in figure 5.16(a), using a point load at the development length of 0.7 in. strands from the end of the girder (15 ft). Although the predicted ultimate flexure capacity of the girder was reached at a load of 780 kips, the girder was able to withstand 800 kips (the capacity of the loading frame) with no significant damage or strand slippage. Therefore, the girder was tested again, as shown in figure 5.16(b), using a point load located at 10 ft from the other end of the girder (shorter than the development length). Again, the girder was able to withstand a load of 800 kips in the second test without significant damage or strand slippage.



(a)



(b)

Figure 5.16 NU900 girder (a) First test, (b) Second test, and (c) Test setup

(c)

Figure 5.16 NU900 girder (a) First test, (b) Second test, and (c) Test setup cont'd

The load-deflection relationships for the NU900 girder in the first and second test are shown in figure 5.17. The maximum load placed on the girder was 800 kips, which provided a total deflection of approximately 2.5 in. and 1.3 in. for the first and second tests respectively.

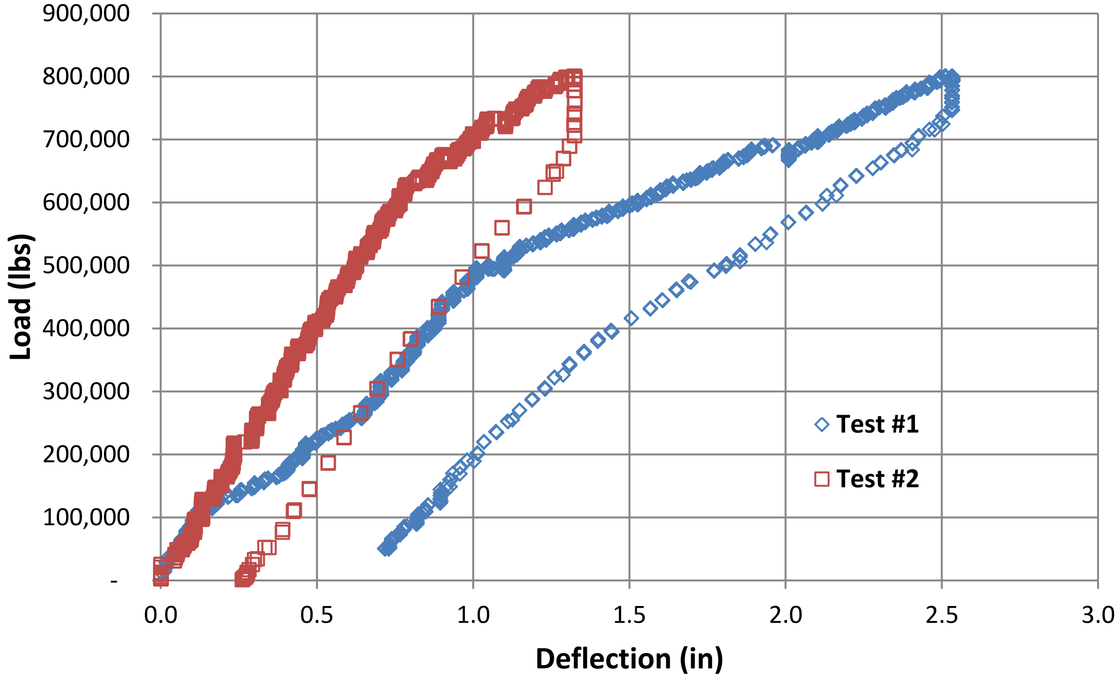


Figure 5.17 Load versus Deflection for the First and Second Tests

Based on the results and observations of the two tests, it was concluded that the NU UHPC mix #5' used in the design and production of the NU900 girders is a very successful mix. The cracking pattern, location, and intensity were normal and predictable in the two tests. Although the girder was not loaded to failure, due to the high compressive strength of the deck concrete and the limitations on the capacity of the loading frame, several conclusions were made: 1) The load-deflection relationships have indicated that the actual cracking moment is very close to the predicted one; 2) Prestressed NU UHPC girders have adequate ductility; 3) NU UHPC has excellent bond strength with prestressing strands that allows the full development of 0.7 in. strands at 2 in by 2 in. spacing with conventional reinforcement details.

Chapter 6 Design and Production Recommendations

Based on the large number of trial mixes designed and tested within this project, the following two mixes are recommended as economical and practical mixes that have performance characteristics superior to those of the mixes currently used in Nebraska for bridge I-girders.

Table 6.1 Components of AASHTO Type II and NU900 Girders Mixes

Component	AASHTO Type II Girder Mix	NU900 Girder Mix
#10 Sand (lb/cy)	2,075	1,580
1/4" BRS (lb/cy)	-	672
Cement Type III (lb/cy)	1,120	1,050
Class C Fly Ash (lb/cy)	240	300
Silica Fume (lb/cy)	240	150
Chryso HRWR (gal/cy)	8	5.0
Cold Water (gal/cy)	29	29
Cost ($/cy)	215	157

The two recommended mixes consist of local materials that are readily available to precast prestressed concrete producers in Nebraska, such as #10 sand, ¼” BRS, type III cement, and class C fly ash. Other materials, such as silica fume, and Chryso Fluid Premia 150 are commercially available and should be ordered in advance. Please visit: us.chryso.com and www.silicafume.org for ordering information.

The two recommended mixes have been implemented in full-scale girder production. Below is the recommended batching and mixing procedures:

1. All granular materials are pre-blended for 2-3 minutes (dry mixing);
2. All water and HRWR are added simultaneously;

3. Mixing continues until adequate flowability is achieved (10 to 15 minutes) depending on the quantity being mixed and the mixer capacity and power;
4. Transport the concrete using a truck mixer and check slump-flow on-site;
5. Add HRWR if needed (average spread less than 22 in.).

Also, it should be noted that the behavior of fresh NU UHPC is different from that of a conventional self-consolidating concrete (SCC) in the following aspects:

1. NU UHPC has very high viscosity, so the time after which concrete spread circle reaches 20 in. (T_{50}) is much longer than it of SCC. The SCC recommended range (2 – 5 sec.) does not apply.
2. Spread diameter should be measured when the concrete stops flowing. Spread diameters more than 30 in. are common and acceptable due to the high stability and segregation resistance of NU UHPC mixes (VSI = 0 or 1).
3. Forms must be properly sealed at the joints and corners to prevent leakage of NU UHPC.
4. NU UHPC generates more heat than SCC. Bed and/or internal concrete temperature should be monitored and kept below 135^{o}F while curing.

The compressive strength testing results have shown that a 12 ksi can be specified as a release strength and 15 ksi as a final strength for the two recommended NU UHPC mixes. Other material tests have indicated that the modulus of elasticity of NU UHPC is lower than predicted when using the current AASHTO LRFD specifications. This may results in higher values of camber, deflection, and prestress losses than predicted. Values of the splitting tensile strength and modulus of rupture were found to be within the range predicted using the current AASHTO LRFD specifications. This results in accurate prediction of cracking load and moment for both shear and flexure loadings. Evaluating the shrinkage using the 2007 AASHTO LRFD method has indicated that the NU UHPC mix #4' (AASHTO type II girder mix) has a significantly higher shrinkage than AASHTO predicted values, while the NU UHPC mix #5' (NU900 girder mix) has only slightly higher shrinkage than the AASHTO predicted values. This behavior needs to be further investigated in order to accurately determine the shrinkage for prestress loss calculations.

Design and production of NU UHPC girders had also shown that conventional design and detailing procedures are applicable and adequate for NU UHPC girders. Therefore, it is highly recommended that the outcomes of this project be implemented in the design and production of prestressed girders for bridge projects in Nebraska. A combination of NU UHPC with 0.7 in. prestressing strands would be an ideal combination that is expected to achieve the highest possible moment capacity by balancing the large force from the 0.7 in. stands at final as well as at release.

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Appendix A Material Properties of NU UHPC

Table A.1 Compressive Strength (ksi)

Time (days)	**NU UHPC #1**				**NU UHPC #2**				**NU UHPC #3**				**NU UHPC #4**				**NU UHPC #5**			
	#1	#2	#3	**Ave.**	#1	#2	#3	**Ave.**	#1	#2	#3	**Ave.**	#1	#2	#3	**Ave.**	#1	#2	#3	**Ave.**
1	11.7	11.3	11.5	**11.5**	10.1	10.4	10.2	**10.2**	13.6	12.8	12.6	**13.0**	13.4	13.4	13.5	**13.4**	12.8	12.6	13.4	**12.9**
3	12.9	12.7	13.4	**13.0**	12.5	12.7	12.2	**12.5**	15.1	15.0	14.8	**15.0**	14.3	14.3	13.8	**14.1**	14.45	14.4	15	**14.6**
7	14.7	14.3	14.5	**14.5**	12.6	13.1	12.8	**12.8**	15.4	14.9	14.5	**14.9**	15.1	15.1	15.6	**15.3**	16.1	16.2	16.6	**16.3**
14	15.2	15.2	15.4	**15.2**	13.7	13.3	13.4	**13.5**	16.1	16.6	16.2	**16.3**	15.1	15.3		**15.2**	15.8	16.3	14.8	**15.6**
28	15.9	15.7		**15.8**	14.6	14.1		**14.3**	17.1	17.3	17.2	**17.2**	16.1	16.0		**16.1**	16.5	17.3		**16.9**

Table A.2 Modulus of Rupture (psi)

NU UHPC # 1				
Specimen	# 1	# 2	# 3	**Average**
Average Depth (in)	5.97	6.05	5.96	**5.99**
Average Width (in)	5.64	5.56	6.09	**5.76**
Load at Failure (lb)	19,530	18,200	18,600	**18,777**
Modulus of Rupture (psi)	**1,749**	**1,610**	**1,548**	**1,635**

NU UHPC # 2				
Specimen	# 1	# 2	# 3	**Average**
Average Depth (in)	5.991	5.99	6.029	**6.00**
Average Width (in)	5.8	6.017	5.977	**5.93**
Load at Failure (lb)	12,350	15,470	13,680	**13,833**
Modulus of Rupture (psi)	**1,068**	**1,290**	**1,133**	**1,164**

NU UHPC # 3				
Specimen	# 1	# 2	# 3	**Average**
Average Depth (in)	6.03	6.01	6.08	**6.04**
Average Width (in)	5.76	5.75	5.75	**5.75**
Load at Failure (lb)	12,290	12,420	11,360	**12,023**
Modulus of Rupture (psi)	**1,056**	**1,076**	**962**	**1,032**

NU UHPC # 4				
Specimen	# 1	# 2	# 3	**Average**
Average Depth (in)	6.01	6.02	6.07	**6.03**
Average Width (in)	5.79	5.78	5.81	**5.79**
Load at Failure (lb)	19,290	16,850	19,560	**18,567**
Modulus of Rupture (psi)	**1,660**	**1,448**	**1,645**	**1,584**

NU UHPC # 5				
Specimen	# 1	# 2	# 3	**Average**
Average Depth (in)	6.13	6.037	6.045	**6.07**
Average Width (in)	5.512	6.185	5.049	**5.58**
Load at Failure (lb)	16,300	15,060	14110	**15,157**
Modulus of Rupture (psi)	**1,417**	**1,203**	**1,377**	**1,332**

Table A.3 Splitting Tensile Strength (psi)

NU UHPC # 1				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	5.98	6.01	5.93	**5.97**
Length (in)	12.01	11.7	11.93	**11.88**
Load (lb)	89,000	102,100	103,800	**98,300**
Splitting Tensile Strength (psi)	**789**	**924**	**934**	**882**

NU UHPC # 2				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	6.027	5.958	6.01	**6.00**
Length (in)	11.846	12.197	11.869	**11.97**
Load (lb)	94,700	98,400	100,500	**97,867**
Splitting Tensile Strength (psi)	**844**	**862**	**897**	**868**

NU UHPC # 3				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	5.99	5.99	5.93	**5.97**
Length (in)	11.9	12.04	11.78	**11.91**
Load (lb)	94,700	106,800	104,700	**102,067**
Splitting Tensile Strength (psi)	**846**	**943**	**954**	**914**

NU UHPC # 4				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	5.99	6.04	6.002	**6.01**
Length (in)	12.208	12.048	11.905	**12.05**
Load (lb)	132,600	127,000	118,400	**126,000**
Splitting Tensile Strength (psi)	**1,154**	**1,111**	**1,055**	**1,107**

NU UHPC # 5				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	6.12	6.04	5.98	**6.05**
Length (in)	11.796	12.048	11.435	**11.76**
Load (lb)	95,200	84,500	83,500	**87,733**
Splitting Tensile Strength (psi)	**840**	**739**	**777**	**785**

Table A.4 Modulus of Elasticity (psi)

NU UHPC # 1				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	6.01	5.93	5.98	**5.97**
Length (in)	11.94	11.93	12.01	**11.96**
Modulus 1 (ksi)	6,550	6,429	6,345	**6,441**
Modulus 2 (ksi)	6,544	6,607	6,564	**6,572**
Modulus 3 (ksi)	6,522	6,257	6,339	**6,373**
Modulus Average (ksi)	**6,539**	**6,431**	**6,416**	**6,462**

NU UHPC # 2				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	5.97	6.043	6.012	**6.01**
Length (in)	12.18	12.09	12.41	**12.23**
Modulus 1 (ksi)	3,809	4,163	4,085	**4,019**
Modulus 2 (ksi)	3,760	4,170	3,994	**3,975**
Modulus 3 (ksi)	3,733	4,170	4,029	**3,977**
Modulus Average (ksi)	**3,767**	**4,168**	**4,036**	**3,990**

NU UHPC # 3				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	5.94	6.01	6.03	**5.99**
Length (in)	11.85	11.91	11.92	**11.89**
Modulus 1 (ksi)	6,658	4,163	4,085	**4,969**
Modulus 2 (ksi)	6,741	4,170	3,994	**4,968**
Modulus 3 (ksi)	6,699	4,170	4,029	**4,966**
Modulus Average (ksi)	**6,699**	**4,168**	**4,036**	**4,968**

NU UHPC # 4				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	5.94	6.02	6.09	**6.02**
Length (in)	11.85	11.93	11.98	**11.92**
Modulus 1 (ksi)	6,117	6,244	6,238	**6,200**
Modulus 2 (ksi)	6,182	6,211	6,195	**6,196**
Modulus 3 (ksi)	6,124	6,126	6,117	**6,122**
Modulus Average (ksi)	**6,141**	**6,194**	**6,183**	**6,173**

NU UHPC # 5				
Specimen	# 1	# 2	# 3	**Average**
Diameter (in)	5.96	6.02	6.01	**6.00**
Length (in)	11.92	11.97	11.91	**11.93**
Modulus 1 (ksi)	5,990	6,010	6,110	**6,037**
Modulus 2 (ksi)	5,930	5,090	6,040	**5,687**
Modulus 3 (ksi)	5,890	6,040	6,070	**6,000**
Modulus Average (ksi)	**5,937**	**5,713**	**6,073**	**5,908**

Table A.5 Length Change (%)

Time (days)	NU UHPC # 1						
	Datum	# 1	# 2	# 3	Average	Difference	**Length Change**
1	0.2947	0.2765	0.2694	0.2811	0.276	0.019	**0.00%**
3	0.3532	0.3316	0.3235	0.3095	0.322	0.032	**0.13%**
7	0.352	0.31	0.2905	0.3185	0.306	0.046	**0.27%**
14	0.351	0.3067	0.3212	0.3263	0.318	0.033	**0.14%**
28	**0.3506**	**0.3087**	**0.3184**	**0.3309**	**0.319**	**0.031**	**0.12%**

Time (days)	NU UHPC # 2						
	Datum	# 1	# 2	# 3	Average	Difference	**Length Change**
1	0.3282	0.2979	0.2841		0.291	0.037	**0.00%**
3	0.3505	0.3136	0.3045		0.309	0.041	**0.04%**
7	0.348	0.3172	0.314		0.316	0.032	**-0.05%**
14	0.3505	0.3204	0.3159		0.318	0.032	**-0.05%**
28	**0.3591**	**0.3294**	**0.3256**		**0.328**	**0.032**	**-0.06%**

Time (days)	NU UHPC # 3						
	Datum	# 1	# 2	# 3	Average	Difference	**Length Change**
1	0.3283	0.4306	0.205	0.2395	0.292	0.037	**0.00%**
3	0.331	0.4491	0.2247	0.251	0.308	0.023	**-0.14%**
7	0.3398	0.4489	0.2146	0.2534	0.306	0.034	**-0.02%**
14	0.3409	0.4432	0.2149	0.2515	0.303	0.038	**0.01%**
28	**0.3416**	**0.451**	**0.2169**	**0.2605**	**0.309**	**0.032**	**-0.04%**

Time (days)	NU UHPC # 4						
	Datum	# 1	# 2	# 3	Average	Difference	**Length Change**
1							
3							
7	0.3487	0.2959	0.4909	0.336	0.374	-0.026	**0.00%**
14	0.3621	0.3041	0.4967	0.3405	0.380	-0.018	**0.07%**
28	**0.3698**	**0.3099**	**0.4993**	**0.3492**	**0.386**	**-0.016**	**0.09%**

Time (days)	NU UHPC # 4						
	Datum	# 1	# 2	# 3	Average	Difference	**Length Change**
1	0.354	0.3596	0.3924		0.376	-0.022	**0.00%**
3	0.361	0.3712	0.3955		0.383	-0.022	**0.00%**
7	0.359	0.3724	0.3968		0.385	-0.026	**-0.04%**
14	0.354	0.3734	0.4003		0.387	-0.033	**-0.11%**
28	**0.3552**	**0.3625**	**0.3987**		**0.381**	**-0.025**	**-0.03%**