Implementation of Ultra-High-Performance Concrete in Long-Span Precast Pretensioned Elements for Concrete Buildings and Bridges

Phase I Report
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Research Team:

Dr. Maher Tadros, PE, Project Manager, is a Distinguished Professor Emeritus of Civil Engineering at the University of Nebraska. He was the principal author of the first Edition of PCI Bridge Design Manual. He has been recognized by the Precast/Prestressed Concrete Institute as a Titan of the Industry, a Fellow of the Institute and a Distinguished Educator. He has received numerous design, teaching, and research awards. Dr. Tadros is the inventor of the popular THiN-Wall insulated wall panel system. He holds 15 other US patents. He is the founder and managing partner of eCon struct USA LLC. In addition to his role as managing partner, he acts as the senior technical advisor, value engineering specialist, and marketer of precast buildings and bridges.

Dr. John Lawler, Materials Manager, is a Principal of Wiss, Janney, Elstner Associates, Inc. of Northbrook, IL. His areas of specialization include concrete materials, particularly ultra-high performance and fiber-reinforced concrete. Dr. Lawler is a past chair of PCI’s Concrete Materials Technology Committee and is currently a member of PCI’s Technical Activities Council. He has consulted on materials selection and design for new structures, with a focus on durability in severe environments, and is leading WJE’s UHPC practice, including UHPC mixture development and implementation at precast plants.
Dr. Voo Yen Lei, Special Team Consultant, is one of the pioneers of commercialization of UHPC. He co-founded Dura® Technology Company in Malaysia in 2006. Since then, the company has designed and built over 130 UHPC bridges in Malaysia and neighboring countries. Dr. Voo is co-chair with Dr. Tadros of a newly formed committee of fib 6.5 TG tasked with developing international design guides for design of UHPC bridges. Dr. Voo has been assisting the research team with various design and precast production tasks.

Mr. Gary Klein, PE, Structural Expert (vertical shear), is Executive Vice-President and Senior Principal of Wiss, Janney, Elstner Associates, Inc. of Northbrook, IL. Mr. Klein specializes in investigation and structural research for buildings and bridges. He was the principal investigator for PCI’s 1985 specially funded research on the behavior of spandrel beams and co-principal investigator for the study of volume change in precast buildings, a five-year PCI study completed in 2009. Beginning in 2007, Mr. Klein and WJE partnered with NC State on three important PCI research projects: 1) developing a rational design procedure for precast slender spandrel beams, 2) behavior of dapped double tees, and 3) punching shear of beam ledges.

Dr. Gregory Lucier, Structural Expert (building products) has managed the Constructed Facilities Laboratory at North Carolina State University since 2008 and also serves as a Research Associate Professor in the Department of Civil, Construction, and Environmental Engineering. Dr. Lucier has been actively involved with a wide variety of research programs and commercial testing contracts, but his work is focused on precast and prestressed concrete. Along with collaborators at NCSU and WJE, Dr. Lucier has successfully completed several high-profile research programs for PCI including programs on slender spandrel beams, dapped double tees, and punching shear of beam ledges. He has completed numerous commercial tests and private research programs for clients within the precast concrete industry and has experience with many aspects of precast concrete design and fabrication. Along with colleagues from the NCSU School of Architecture, Dr. Lucier also teaches a Precast Design Studio funded by the PCI Foundation.

Dr. George Morcous, PE, Structural Expert (bridge products) is a professor in the department of construction engineering at the University of Nebraska – Lincoln since January 2005. His expertise includes developing innovative use of high performance materials and precast concrete components/systems in bridge construction. Dr. Morcous led NCHRP 12-16 for developing guidelines for the use of self-consolidating concrete (SCC) in cast-in-place bridge components. He led several Nebraska Department of Transportation (NDOT) projects for developing non-proprietary UHPC mixes, design and construction of pre-tensioned concrete beams using 0.7 in. diameter strands, and development and implementation of innovative precast concrete deck systems. Dr. Morcous holds 3 US patents.

Dr. Amgad Girgis, PE, Structural Expert (bridge design) is the Manager of the Bridge division of eConstruct USA LLC and a part-owner of the company. Prior to joining eConstruct, he worked for Kiewit Engineering in Omaha, NE for three years. Prior to that he was a Research Assistant Professor at the University of Nebraska, where he was the principal investigator in several national and state-sponsored projects. Dr. Girgis holds a BSc and MSc in Civil Engineering from Cairo University, Egypt, and a Ph.D. in Structural Engineering from the University of Nebraska. He has twenty-five years of work experience in the field of structural engineering, including bridges and high-rise buildings. Dr. Girgis has served on ACI, PCI, ASCE and AREMA committees. His work has been published in the PCI Journal and other reputable journals. He is also contributing author in the PCI Bridge Design Manual and the PCI State-of-the-Art Report on Full Depth Bridge Precast Deck Panels. Dr. Girgis is also the winner of the Portland Cement Association (PCA) 2001 National Concrete Bridge Design Competition.
Dr. Micheal Asaad, PE, Structural Expert (precast concrete) is a structural engineer with eConstruct USA LLC. Dr. Asaad joined eConstruct team in January 2015. He has worked in the area of bridge research and design since 2013. Thirteen years of structural design, research, and teaching have constituted his technical skills to solve many design and constructability issues related to bridge and building construction. Dr. Asaad has been awarded a couple awards from ACI and ASTM due to his contribution to the concrete industry. Dr. Asaad has participated in a couple of projects funded by the National Cooperative Highway Research Program (NCHRP) to develop guidelines for using self-consolidating concrete (SCC) and full-depth precast decks in bridge construction.

Mr. David Gee, EIT, is a Structural Engineering Analyst and Assistant Project Manager for eConstruct USA, LLC. He holds a master’s degree from the University of Nebraska. His research has focused on optimization of joints between full depth precast pretensioned deck panels by adjusting the longitudinal post-tensioning levels to fully eliminate projecting rebars into the joint. Mr. Gee is responsible for assisting Dr. Tadros and Dr. Lawler in coordinating the design, analysis, specimen production, testing, meetings, and reporting.

Mr. Atilla Kurt is a graduate research assistant in structural engineering at North Carolina State University. Mr. Kurt has worked, at the Constructed Facilities Laboratory (CFL), on multiple project pertaining to precast, prestressed concrete. He has also worked for structural inspection and design companies in Raleigh, NC. Mr. Kurt is responsible for designing and building test setups, performing analysis, and testing bridge and building components for the PCI project at the CFL.

Mr. Adam Sevenker, PE, Structural Expert (precast concrete) is a structural engineer for eConstruct USA LLC and has worked in the area of bridge design since 2013. He holds a master’s degree from the University of Nebraska. He has experience with design, production and erection of precast, prestressed concrete; including the design of precast I-beams, tub beams and segmental bridges. He was the primary designer of the UHPC decked I-beam bridge, planned to be built in Ontario, Canada in 2020.

Dr. Elizabeth Wagner, PE, Materials Expert, is a materials engineer with Wiss, Janney, Elstner Associates, Inc. Since joining WJE in 2016, she has been involved in numerous projects involving field, laboratory, and analytical investigations of concrete materials, bridge decks, and structures. Her background and interests include durability of concrete materials, service life modeling of concrete structures, and development and testing of construction materials. She has assisted in the development, testing, and troubleshooting of UHPC mixtures for precast products, and has provided guidance to departments of transportation developing materials specifications for UHPC.

Mr. Steve Zimmerman, PE, Structural Testing Expert, is Unit Manager and Associate Principal in WJE’s Structural Testing and Instrumentation group in Northbrook, IL. Mr. Zimmerman is registered as a Professional Engineer in Illinois and Iowa. Mr. Zimmerman joined WJE in 2001 and has since been involved in projects investigating wood, steel, and concrete structures as well as laboratory and field testing of construction materials and structural systems. Assignments have included field investigation, assessment, analysis, and repair design of a number of existing precast concrete structures.
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- WJE
- CFL
- N

Participating Precast Concrete Companies:

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1 INTRODUCTION

1.1 Problem Statement
Ultra-high-performance concrete (UHPC) is a new class of concrete that relies on a highly refined microstructure and fiber reinforcement to achieve superior performance characteristics, including high compressive strength, post-cracking tensile strength and ductility, and exceptional long-term durability in aggressive environments. Despite these desirable performance characteristics, applications of UHPC in the United States have been primarily limited to joints between precast bridge deck panels and a small number of demonstration projects in states such as Iowa and New York. A shortage of national design guidelines has resulted in reluctance by designers and owners to accept and utilize this material in more widespread applications, such as precast pretensioned elements for bridge and building projects.

1.2 Objective
The primary objective of this research project is to engage in an organized research and development program that culminates in the introduction of UHPC for precast pretensioned products in bridge and building applications. This objective is advanced through the generation of guidelines for developing and producing UHPC mixtures, a guide specification for UHPC materials qualification and acceptance, and design guidelines with fully-detailed design examples.

1.3 Scope of Work
The scope of the research project consists of three primary Phases designed to meet the stated objectives:

Phase I consists of a detailed review of the existing literature to produce guidelines for developing, implementing, and testing UHPC, and for designing precast pretensioned elements with UHPC. In support of this phase, UHPC mixtures were developed for five participating precast plants using materials local to each plant. These mixtures were implemented at each plant using the plants’ existing batching equipment, while a sixth participating precast plant produced UHPC based on a previously developed mixture, and a variety of UHPC structural elements were produced, with designs based on the preliminary design guidelines developed for this project. Additional preliminary designs for long-span elements for bridges and buildings were also developed, addressing all applicable design limit states, materials, production, and construction considerations revealed through the literature review and plant implementation trials. Deliverables from this Phase include this Phase I report and drafts of the Guidelines for Production of Ultra-High-Performance Concrete (UHPC), UHPC Materials Guide Specification, and design examples, included as attachments to this report.

The Phase I report also provides testing plans for Phase II. The final details of the test program are dependent on the findings addressed in the Phase I report, as well as the input from the PCI Advisory Committee. This program was generated to provide validation for the analytical modeling results and design recommendations in this research. It is proposed to include component testing and full beam testing in Phase II. Component specimens will be developed to test a wide range of variables affecting interface shear to minimize the need for excessive full beam testing. These tests will allow for investigation of the significant parameters at a relatively low cost. Existing data collected from literature was used to enhance the validity of the conclusions and recommendations for interface and vertical shear, the two critical design components in urgent need for guidelines. Medium scale beam specimens are proposed to validate the structural performance obtained from the component testing, and to study issues of constructability and composite action. The following subsections present the anticipated test specimens, which are subject to change, as advice is received from the PCI Advisory Committee.

Additionally, the Phase I report presents some pilot testing results. These tests were conducted on structural elements or components, which were cast during the trial batches by each participant precaster. The analysis of these results will be included in Phase II.

Phase II furthers the design concepts developed during Phase I of this project and culminates in the structural testing of the beams and structural components designed during Phase I. Detailed finite element analyses, strut-and-tie analyses, and other analytical methods will be used to support the experimental program to provide the information necessary to implement UHPC in practice. Deliverables from this Phase will include the Phase II report,
Introduction

A draft of the Guidelines for Design, and revised design examples formatted for possible inclusion in the PCI Bridge Design Manual (bridge beams) and the PCI Industry Handbook (building beams).

Phase III consists of technology transfer and demonstration projects. In addition to the guide documents developed as part of Phases I and II of this research effort, additional educational webinars and demonstration activities by participating precasters will be conducted to promote more widespread usage of UHPC in precast pretensioned applications.

1.4 Outline of Report

This report presents the research completed as part of Phase I of this project and presents a refined research plan for Phase II. The report is organized as follows:

- Chapter 2 summarizes the UHPC mixture development and implementation activities completed at the six participating precast plants.
- Chapters 3 to 9 present the structural behavior of UHPC under different stress states including tensile, flexure, bond, vertical shear, interface shear, and punching shear behavior. This includes previous work, proposed recommendations for structural design, and plans for testing for Phase II.
- Chapter 10 addresses miscellaneous topics affecting the structural design and construction.
- Chapter 11 summarizes the process of developing new or modified section for UHPC products.
- Appendix A provides guidelines for UHPC production
- Appendix B addresses the materials specifications for UHPC
- Appendix C provides the mill reports from precasters, including materials used with their accompanying specifications
- Appendix D provides precasters’ material characterization
- Appendix E presents a design example for parking garage
- Appendix F presents a design example for 250-ft-span bridge
2 UHPC MIXTURE DEVELOPMENT, IMPLEMENTATION, AND CHARACTERIZATION

The first task of this research project consisted of the development, implementation, and characterization of UHPC mixtures to be used by individual precasters to produce long-span precast pretensioned UHPC elements. An important objective of this Phase of the research project was to facilitate the development and implementation of UHPC by the precasters through cost-effective and readily-implemented processes. This objective was advanced by prioritizing the use of locally-available raw materials and the plants' existing production facilities whenever possible.

To meet this objective, a comprehensive review of published Codes, Specifications, research, and guide documents pertaining to the production and implementation of UHPC was conducted. Based on this review of the literature, the experience of the project team, and discussions with other UHPC practitioners, efficient methods for developing, implementing, and characterizing UHPC mixtures within the scope of this research project were developed. These methods were demonstrated and refined through laboratory mixture development programs and field production trials for each of six participating precast plants, as documented in the subsequent sections of this report.

To support future development and implementation of UHPC in precast applications, two documents, Guidelines for Production of Ultra-High-Performance Concrete (UHPC) and a UHPC Materials Guide Specification, were drafted. The Guidelines for Production of Ultra-High-Performance Concrete (UHPC) summarizes the state-of-the-art in UHPC production and provides guidance to precasters and owners who are considering UHPC for precast, pretensioned applications. The UHPC Materials Guide Specification provides mandatory language related to material-issues specific to UHPC and defines the performance characteristics and production requirements necessary for construction of structural UHPC elements. This specification was written such that the language could be incorporated into either bridge or building projects and relies on PCI MNL-116 Manual for Quality Control for Plants and Production of Structural Precast Concrete Products by reference where possible. Draft versions of these documents are provided in Appendices A and B, respectively, of this report, and final versions, incorporating additional feedback from the participating precasters, will be provided as a deliverable upon completion of this Project.

2.1 UHPC Mixture Development in Laboratory

New UHPC mixtures were developed for five of the participating precasters, Precasters A, B, C, D, and E. The sixth precaster, Precaster F, had previously developed a UHPC mixture independent of this Project. The laboratory mixture development process and results for each precaster are presented below. This follows the process outlined in Section A3.2.2.3 of the Guidelines for Production of UHPC.

2.1.1 Development Process

The UHPC mix development process consisted of four steps for each precaster: (1) materials review and analysis, (2) preliminary mixture development, (3) laboratory workability trials, and (4) compressive strength evaluation.

Step 1. Materials Review and Analysis

Each precaster submitted mill certificates, gradations, and technical data sheets, as available, for locally available materials that may be considered viable candidates for UHPC. Preference was given to the materials currently used by the plants, but additional materials available from local suppliers were also considered. The following guidance was provided to the producers to guide the selection of candidate materials:

- **Cement**: Type I, Type II, or Type V cement with a C₃A content less than 8%, a moderate or low heat of hydration, and a Blaine fineness less than 400 m²/g were preferred. White cements were acceptable, provided they met the C₃A and Blaine fineness recommendations. Cements not meeting these criteria were permitted, but plants were cautioned that materials not meeting these criteria may lead to challenges with respect to initial workability and workability retention.

- **Silica Fume**: Silica fume with a SiO₂ content greater than 95% was preferred. Silica fume was required for each mixture, regardless of whether the plant currently uses silica fume for other precast applications.

- **Supplemental Material**: Up to two candidate supplemental materials were permitted for preliminary evaluation, but were not required to be included in the mixture. Possible supplemental materials included
Implementation of UHPC in Long-Span Precast Pretensioned Elements

UHPC Mixture Development, Implementation, and Characterization

fly ash, slag, ground silica, limestone powder, and metakaolin. The material was recommended to have an average particle size between cement and silica fume, if possible, but materials of other sizes were also considered.

- **Sand**: Finer sands, with a maximum particle size less than 0.03 inches (No. 20 sieve) were recommended. The sand was required to meet the deleterious materials requirements specified in ASTM C33 or C114.

- **High-Range Water Reducer (HRWR)**: A liquid HRWR with a high efficiency in high-powder mixtures was recommended. It was required that the admixture not have a deleterious impact on the concrete performance at dosages above the manufacturer’s recommended levels for conventional concrete. Producers were encouraged to consult their local admixture suppliers for product recommendations.

Samples of the candidate materials were obtained from each of the producers for additional characterization and review. The particle size distributions of the powder materials were determined by laser particle size analysis using a Malvern Mastersizer 2000. Laser diffraction was performed using wet dispersion methods, with samples dispersed in isopropanol to minimize hydration during testing. Due to the difficulty of fully dispersing the silica fume agglomerates, a “representative” silica fume distribution was assumed based on distributions provided by the Elkem Materials - Mixture Analyzer (EMMA) software program (Elkem Materials, 2016).

**Step 2. Preliminary Mixture Development**

Candidate mixture proportions were determined by comparing various combinations of cement, silica fume, supplemental material, and sand to a target gradation based on the modified Andreasen and Andersen (A&A) model. The modified A&A model is defined by the equation:

\[
P(D) = \frac{D^q - D_{\text{min}}^q}{D_{\text{max}}^q - D_{\text{min}}^q}
\]

where \( P \) is the percent of particles with a diameter smaller than \( D \); \( D_{\text{min}} \) and \( D_{\text{max}} \) are the minimum and maximum particle diameters, respectively; and \( q \) is a shape parameter between 0 and 1. For context, a shape parameter, \( q \), value closer to 1 produces a more coarsely graded mixture, and a \( q \) value closer to 0 produces a more finely graded mixture; typical \( q \) values selected for UHPC mixtures are in the range of 0.19 to 0.37. (See Section A3.2.2.3 of the Guidelines for Production of UHPC for more details of the modified A&A model.)

For preliminary mixture identification, \( D_{\text{min}} \) and \( D_{\text{max}} \) were selected based on the minimum and maximum particle sizes in the grouping of materials considered, respectively, and \( q \) was selected to be 0.30 or 0.25. A spreadsheet-based optimization algorithm was developed to identify the relative proportions of materials for each combination selected that minimized the mismatch between the combined gradation and the A&A “ideal” target, with the mismatch described by an “error” term based on the sum of squared errors (SSE) for each particle size. While the error has meaning only for comparative purposes, errors of about 150 or less generally indicated a good match between optimized and ideal gradation. To promote good overall UHPC performance and economy, the total silica fume content was limited to no more than 25 percent by weight of cement, the total supplemental material was limited to no more than 30 percent by weight of cement, and the sand-to-cement ratio was limited to no more than 2:1.

Candidate mixture combinations, each having the lowest “error” for the particular combination of materials considered, were identified and presented to each precaster. The precasters then selected one candidate mixture design for further development based on consideration of material availability, quality, and cost.

**Step 3. Laboratory Workability Trials**

A series of laboratory trial batches were produced based on the selected mixture design. The purpose of the laboratory trial batching was to identify the minimum water content and HRWR content for the UHPC mixture expected to produce a flow spread between 8 and 11 inches in plant production. Because the laboratory mixer will differ in mixing energy from a plant mixer, a target flow spread of 9 ± 0.25 inches was selected for the laboratory trial batches. All laboratory trial batches were prepared without fibers.

The trial batches were prepared in 40-cubic inch volumes using a 6-quart benchtop mixer. The basic mixing protocol used for each batch is outlined below:

1. Dry-mix the sand and silica fume for 5 minutes to break up agglomerates of silica fume.
2. Add the cement and supplemental materials (if used), and dry-mix for an additional 2 minutes at medium speed.
3. Gradually add 2/3 of the mixing water over a 1-minute interval.
4. Gradually add the remaining mixing water and all of the HRWR over the next 1-minute interval.
5. Continue mixing at medium speed until the mixture turns into a fluid state (about 10 to 20 minutes for the laboratory mixer).
6. Increase the mixer speed to high and mix for an additional 3 minutes.
7. Stop the mixer and measure flow spread according to ASTM C1856.

A modified mixing protocol was used for mixtures containing Chryso Premia 150 HRWR, based on recommendations from the admixture supplier. In this modified protocol, all of the mixing water was added in Step 3, and the material was mixed for 5 minutes. Then all of the HRWR was added gradually over 1 minute as Step 4. All other steps in the mixing protocol remained the same.

During the trial mixture development, adjustments were made to the water-binder ratio (w/b), HRWR dosage, HRWR type, and particle packing parameter q, to achieve the lowest w/b and HRWR dosage that achieved the target workability. Variables generally were varied within the ranges identified in Table 2.1.1-1 below.

Table 2.1.1-1. Mixture Design Factors Varied During Workability Trials

<table>
<thead>
<tr>
<th>Factor</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>HRWR Type</td>
<td>Current product; Alternate product (if available)</td>
</tr>
<tr>
<td>w/b</td>
<td>Target = 0.20 or less</td>
</tr>
<tr>
<td>HRWR Dosage</td>
<td>Target = Minimized, Max. = 6% by weight of binder</td>
</tr>
<tr>
<td>q</td>
<td>Start = 0.30 (Range = 0.19 to 0.37)</td>
</tr>
</tbody>
</table>

Step 4, Compressive Strength Evaluation

Once an “optimized” mixture design was determined, a set of three 2-inch cubes was prepared for compressive strength evaluation (again, without fibers). The purpose of this evaluation was to verify that the mixture had the potential to achieve at least 18,000 psi compressive strength, as required by the design guidelines, rather than to evaluate the actual “production” compressive strength of the mixture as might be measured in testing 3x6-inch cylinders. The smaller 2-inch cube geometry was selected based on the limited amount of material produced with each batch (40 cubic inches), and an accelerated heat curing process was employed to facilitate as much strength development as possible over a relatively short duration. The three cubes were prepared by filling the molds in a single lift, then consolidating the top lift according to ASTM C109 Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens). The cubes were cured in their molds for 24 to 48 hours, then placed into a water bath at 194 °F (90 °C) for 48 hours essentially according to the accelerated post-curing heat treatment specified for larger specimens under ASTM C1856, Standard Practice for Fabricating and Testing Specimens of Ultra-High-Performance Concrete. The cubes were then slowly cooled to room temperature (70 °F) and placed into a 100% relative humidity environment for 24 hours, then tested according to ASTM C109, with a modified loading rate of 145 psi per second.

2.1.2 UHPC Mixtures Developed

The five UHPC mixtures developed according to this process are summarized for each plant in the following sections. The mixture proportions do not consider air content in volume calculations. Mill certificates, aggregate gradations, and technical data sheets for materials used in the final mixture designs are included in Appendix C. Particle size distributions determined by laser particle size analysis are included in Appendix D.

2.1.2.1 Precaster A

Based on discussions with plant personnel, the candidate materials and preliminary mixture proportions listed in Table 2.1.2-1-1 were selected for initial laboratory trial batching for Precaster A. Preliminary mixture proportions were based on an assumed q value of 0.30. The materials generally met the recommendations listed above, with the exception of the Type III cement, which had a Blaine fineness of 533 m²/g, greater than the recommended limit of 400 m²/g. Preliminary mixture development (trial batching) was based on the current HRWR used by Precaster A, GCP Advacast 575, with alternative HRWRs used as needed.
A series of 25 laboratory trial batches were produced based on the selected candidate mixture design, varying the parameters listed in Table 2.1.1.1 until the minimum w/b and HRWR dosage achieving a flow spread of 9 ± 0.25 inches was identified. A summary of each mixture tested and the adjustments made between batches is presented in Table 2.1.2.1-2. During trial batching, it was observed that the Type III cement mixture appeared to be more sensitive to total moisture content and to have a greater water demand than UHPC mixtures developed for other precasters with Type I/II cements. It was also observed that greater flow could be achieved for the same w/b and HRWR dosages when the "q" shape parameter was reduced from 0.30 to 0.25 (i.e., when the mixture was more finely graded). Sufficient workability could not be achieved with the mixture proportions tested using the plant's current HRWR, Adva Cast 575; an alternative admixture was needed. Of the HRWRs tested, the best workability was achieved using the Chryso Premia 150 HRWR at a dosage of 80.5 fl. oz./cwt of binder (Trial #24).
### Table 2.1.2.1-2. Trial Batch Summary - Precaster A

<table>
<thead>
<tr>
<th>Trial</th>
<th>q</th>
<th>SF/Cement Ratio</th>
<th>Sand/Cement Ratio</th>
<th>w/b</th>
<th>HRWR Type</th>
<th>HRWR (fl. oz./cwt binder)</th>
<th>Flow Spread (in.)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.167</td>
<td>Adva Cast 575</td>
<td>69</td>
<td>--</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>2</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.175</td>
<td>Adva Cast 575</td>
<td>69</td>
<td>--</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>3</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.183</td>
<td>Adva Cast 575</td>
<td>69</td>
<td>--</td>
<td>Did not flow; increase w/b and HRWR</td>
</tr>
<tr>
<td>4</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.197</td>
<td>Adva Cast 575</td>
<td>92</td>
<td>--</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>5</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.221</td>
<td>Adva Cast 575</td>
<td>92</td>
<td>--</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>6</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.246</td>
<td>Adva Cast 575</td>
<td>92</td>
<td>4.0</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>7</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.262</td>
<td>Adva Cast 575</td>
<td>92</td>
<td>4.8</td>
<td>Flow less than target; increase HRWR</td>
</tr>
<tr>
<td>8</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.263</td>
<td>Adva Cast 575</td>
<td>138</td>
<td>4.6</td>
<td>Flow decreased; decrease HRWR</td>
</tr>
<tr>
<td>9</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.261</td>
<td>Adva Cast 575</td>
<td>46</td>
<td>6.1</td>
<td>Flow increased but less than target; decrease HRWR</td>
</tr>
<tr>
<td>10</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.260</td>
<td>Adva Cast 575</td>
<td>23</td>
<td>--</td>
<td>Did not flow; w/b above target limit; try alternate HRWR</td>
</tr>
<tr>
<td>11</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.260</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>10.1</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>12</td>
<td>0.30</td>
<td>0.23</td>
<td>1.41</td>
<td>0.244</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>8.8</td>
<td>Flow at target; w/b above target max.; try alternative q</td>
</tr>
<tr>
<td>13</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.240</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>10.1</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>14</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.224</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>8.9</td>
<td>Flow at target; increase HRWR</td>
</tr>
<tr>
<td>15</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.224</td>
<td>MasterGlenium 7920</td>
<td>57</td>
<td>9.0</td>
<td>Flow unchanged; decrease HRWR</td>
</tr>
<tr>
<td>16</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.224</td>
<td>MasterGlenium 7920</td>
<td>40</td>
<td>8.9</td>
<td>Flow unchanged; w/b above target; try alternate HRWR</td>
</tr>
<tr>
<td>17</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.224</td>
<td>Premia 150</td>
<td>23.5</td>
<td>--</td>
<td>Did not flow; increase HRWR</td>
</tr>
<tr>
<td>18</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.224</td>
<td>Premia 150</td>
<td>46</td>
<td>9.4</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>19</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.215</td>
<td>Premia 150</td>
<td>46</td>
<td>6.7</td>
<td>Flow less than target; increase w/b and HRWR</td>
</tr>
<tr>
<td>20</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.224</td>
<td>Premia 150</td>
<td>69</td>
<td>10.5</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>21</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.208</td>
<td>Premia 150</td>
<td>69</td>
<td>9.5</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>22</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.192</td>
<td>Premia 150</td>
<td>69</td>
<td>--</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>23</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.200</td>
<td>Premia 150</td>
<td>69</td>
<td>8.0</td>
<td>Flow less than target; increase HRWR</td>
</tr>
<tr>
<td>24</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.200</td>
<td>Premia 150</td>
<td>80.5</td>
<td>8.6</td>
<td>Flow at target; increase HRWR; evaluate strength</td>
</tr>
<tr>
<td>25</td>
<td>0.25</td>
<td>0.25</td>
<td>1.10</td>
<td>0.200</td>
<td>Premia 150</td>
<td>93</td>
<td>8.4</td>
<td>Flow decreased; Trial #24 is at optimal proportions.</td>
</tr>
</tbody>
</table>
The mixture recommended for in-plant trial production had a w/b of 0.20, and consisted of Type III cement, 25 percent silica fume by weight of cement, and fine sand at a sand/cement ratio of 1.10 by weight. The average compressive strength of three 2-inch cubes prepared from this mixture (without fibers) and cured and tested as described above, was found to be 27,820 psi, which was estimated to correspond to a compressive strength of approximately 25,000 psi for 3-inch by 6-inch cylinders. Therefore, it was predicted that the UHPC mixture designed for Precaster A (Table 2.1.2.1-3) has the potential to achieve a minimum compressive strength of 18,000 psi during field production.

Table 2.1.2.1-3. Recommended Mixture Design for In-Plant Trials - Precaster A

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Amount (lb./yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type III Cement (Lehigh, Bellingham, WA)</td>
<td>1514</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>Force 10,000D Microsilica (GCP)</td>
<td>378</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>None</td>
<td>--</td>
</tr>
<tr>
<td>Sand</td>
<td>-50 Sand, #8750 (CalPortland, DuPont, WA)</td>
<td>1665</td>
</tr>
<tr>
<td>Water</td>
<td>--</td>
<td>304 (36.4 gal.)</td>
</tr>
<tr>
<td>High-Range Water Reducer</td>
<td>Chryso Premia 150</td>
<td>105 (1523 fl. oz.)</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>Steel Fiber (Dura)</td>
<td>263</td>
</tr>
</tbody>
</table>

2.1.2.2 Precaster B

Based on discussions with plant personnel, the candidate materials and preliminary mixture proportions listed in Table 2.1.2.2-1 were selected for initial laboratory trial batching for Precaster B. Preliminary mixture proportions were based on an assumed q value of 0.25. All materials met the recommendations listed above, with the exception that the limestone powder was slightly coarser than the cement. All preliminary mixture development (trial batching) was based on the current HRWR used by Precaster B, BASF MasterGlenium 7920.

Table 2.1.2.2-1. Candidate Mixture - Precaster B

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Relative Proportion, by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II Cement (Argos, Harleyville, SC)</td>
<td>1.00</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>BASF MasterLife SF 100</td>
<td>0.25</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>Imerfill 25 Limestone Powder (Imerys)</td>
<td>--</td>
</tr>
<tr>
<td>Sand</td>
<td>Mason Sand (Martin Marietta, Calhoun Quarry, SC)</td>
<td>1.37</td>
</tr>
<tr>
<td>Error (q = 0.25)</td>
<td>--</td>
<td>116</td>
</tr>
</tbody>
</table>

A series of 12 laboratory trial batches were produced based on the selected candidate mixture design, varying the parameters listed in Table 2.1.1-1 until the minimum w/b and HRWR dosage achieving a flow spread of 9 ± 0.25 inches was identified. A summary of each mixture tested and the adjustments made between batches is presented in Table 2.1.2.2-2. It was observed that the greatest flow could be achieved when the “q” shape parameter was reduced from 0.25 to 0.23. The best performance was achieved for a silica-fume only mixture containing the MasterGlenium 7920 HRWR at a dosage of 46 fl. oz./cwt of binder (Trial #11). After the mixture proportions had been optimized without a supplemental material, one additional trial batch (Trial #12) was produced at the precaster’s request containing limestone powder as a supplemental material. This trial batch was prepared at the same q-value and nominal w/b and HRWR dosage as the previously optimized batch. Slightly improved workability was observed for this mixture relative to the mixture containing silica fume only; therefore, this mixture was selected for field implementation, although it was not optimized relative to minimum w/b, HRWR dosage, and “q” factor.

---

1 Graybeal and Davis (2008) report that the average compressive strength of 3-inch by 6-inch UHPC cylinder is approximately equal to 1.00 times the compressive strength of a 2-inch cube of the same material. Limited testing for this project indicates that the compressive strength of a 3-inch by 6-inch UHPC cylinder is approximately 0.90 times the compressive strength of a 2-inch cube of the same material. Therefore, for the purposes of establishing whether a mixture can achieve the minimum specified cylinder compressive strength, the compressive strength of 3-inch by 6-inch cylinders was conservatively estimated to be 0.90 times the compressive strength measured on 2-inch cubes.
### Table 2.1.2.2-2. Trial Batch Summary - Precaster B

<table>
<thead>
<tr>
<th>Trial</th>
<th>q</th>
<th>SF/Cement Ratio</th>
<th>LS/Cement Ratio</th>
<th>Sand/Cement Ratio</th>
<th>w/b</th>
<th>HRWR Type</th>
<th>HRWR (fl. oz./cwt binder)</th>
<th>Flow Spread (in.)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.37</td>
<td>0.216</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>10.8</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.37</td>
<td>0.200</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>9.8</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>3</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.37</td>
<td>0.192</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>9.1</td>
<td>Flow at target; consider alternate q</td>
</tr>
<tr>
<td>4</td>
<td>0.30</td>
<td>0.20</td>
<td>0</td>
<td>1.70</td>
<td>0.200</td>
<td>MasterGlenium 7920</td>
<td>47</td>
<td>9.2</td>
<td>Flow unchanged; consider lower q</td>
</tr>
<tr>
<td>5</td>
<td>0.23</td>
<td>0.25</td>
<td>0</td>
<td>1.22</td>
<td>0.192</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>9.6</td>
<td>Flow increased above target; reduce w/b</td>
</tr>
<tr>
<td>6</td>
<td>0.23</td>
<td>0.25</td>
<td>0</td>
<td>1.22</td>
<td>0.184</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>9.2</td>
<td>Flow near target; reduce w/b</td>
</tr>
<tr>
<td>7</td>
<td>0.23</td>
<td>0.25</td>
<td>0</td>
<td>1.22</td>
<td>0.176</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>8.2</td>
<td>Flow less than target; increase w/b and decrease HRWR</td>
</tr>
<tr>
<td>8</td>
<td>0.23</td>
<td>0.25</td>
<td>0</td>
<td>1.22</td>
<td>0.184</td>
<td>MasterGlenium 7920</td>
<td>40</td>
<td>8.9</td>
<td>Flow near target; decrease HRWR</td>
</tr>
<tr>
<td>9</td>
<td>0.23</td>
<td>0.25</td>
<td>0</td>
<td>1.22</td>
<td>0.176</td>
<td>MasterGlenium 7920</td>
<td>57</td>
<td>8.3</td>
<td>Flow less than target; increase w/b and decrease HRWR</td>
</tr>
<tr>
<td>10</td>
<td>0.23</td>
<td>0.25</td>
<td>0</td>
<td>1.22</td>
<td>0.184</td>
<td>MasterGlenium 7920</td>
<td>52</td>
<td>9.0</td>
<td>Flow at target; Trial #6 optimal proportions; re-batch Trial #6</td>
</tr>
<tr>
<td>11</td>
<td>0.23</td>
<td>0.25</td>
<td>0</td>
<td>1.22</td>
<td>0.184</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>8.9</td>
<td>Flow consistent with previous trials; evaluate strength</td>
</tr>
<tr>
<td>12</td>
<td>0.23</td>
<td>0.25</td>
<td>0.11</td>
<td>1.45</td>
<td>0.184</td>
<td>MasterGlenium 7920</td>
<td>46</td>
<td>9.2</td>
<td>Added limestone powder at precaster request; mixture not optimized</td>
</tr>
</tbody>
</table>
The mixture recommended for in-plant trial production had a w/b of 0.184, and consisted of Type I/II cement, 25 percent silica fume by weight of cement, 11 percent limestone powder by weight of cement, and fine sand at a sand/cement ratio of 1.45 by weight. The average compressive strength of three 2-inch cubes prepared from this mixture (without fibers) and cured and tested as described above, was found to be 26,020 psi, which was estimated to correspond to a compressive strength of approximately 23,400 psi for 3-inch by 6-inch cylinders. Therefore, it was predicted that the UHPC mixture designed for Precaster B (Table 2.1.2.2-3) has the potential to achieve a minimum compressive strength of 18,000 psi during field production.

Table 2.1.2.2-3. Recommended Mixture Design for In-Plant Trials - Precaster B

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Amount (lb./yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II Cement (Argos, Harleyville, SC)</td>
<td>1,297</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>BASF MasterLife SF 100</td>
<td>324</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>Imerfill 25 Limestone Powder (Imerys)</td>
<td>143</td>
</tr>
<tr>
<td>Sand</td>
<td>Mason Sand (Martin Marietta, Calhoun Quarry, SC)</td>
<td>1,881</td>
</tr>
<tr>
<td>Water</td>
<td>--</td>
<td>288 (34.5 gal.)</td>
</tr>
<tr>
<td>High-Range Water Reducer</td>
<td>BASF MasterGlenium 7920</td>
<td>57 (811 fl. oz.)</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>Steel Fiber (Dura)</td>
<td>263</td>
</tr>
</tbody>
</table>

2.1.2.3 Precaster C

Based on discussions with plant personnel, the candidate materials and preliminary mixture proportions listed in Table 2.1.2.3-1 were selected for initial laboratory trial batching for Precaster C. Preliminary mixture proportions were based on an assumed q value of 0.25. The materials generally met the recommendations listed above, with the exception of the Type I/II cement, which had a Blaine fineness of 412 m²/g, slightly greater than the recommended limit of 400 m²/g. Preliminary mixture development (trial batching) was based on the current HRWR used by Precaster C, BASF MasterGlenium 7920, and included a rheology-modifying admixture, GCP V-MAR F100, which had been used by the plant in prior UHPC mixtures to improve workability and product finishability at elevated ambient temperatures.

Table 2.1.2.3-1. Candidate Mixture - Precaster C

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Relative Proportion, by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II Cement (Titan, Patras, Greece)</td>
<td>1.00</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>BASF MasterLife SF 100</td>
<td>0.25</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>None</td>
<td>--</td>
</tr>
<tr>
<td>Sand</td>
<td>Mason Sand (ER Jahna, Independent North Mine)</td>
<td>0.97</td>
</tr>
<tr>
<td>Error (q = 0.25)</td>
<td>--</td>
<td>210</td>
</tr>
</tbody>
</table>

A series of 11 laboratory trial batches were produced based on the selected candidate mixture design, varying the parameters listed in Table 2.1.1-1 until the minimum w/b and HRWR dosage achieving a flow spread of 9 ± 0.25 inches was identified. After the target flow spread was achieved for mixes containing HRWR as the only chemical admixture, a limited series of tests were also performed in which the rheology-modifying admixture (RMA) dosage was incrementally increased and the surface characteristics of the UHPC were observed. A summary of each mixture tested and the adjustments made between batches is presented in Table 2.1.2.3-2. The target performance was achieved for a silica-fume only mixture containing the MasterGlenium 7920 HRWR at a dosage of 43 fl. oz./cwt of binder and the V-MAR F100 RMA at a dosage of 6 fl. oz./cwt of binder (Trial #11). Reducing the “q” shape parameter below 0.25 did not have a notable impact on the performance of the mixture.
### Table 2.1.2.3-2. Trial Batch Summary - Precaster C

<table>
<thead>
<tr>
<th>Trial</th>
<th>q</th>
<th>SF/Cement Ratio</th>
<th>Sand/Cement Ratio</th>
<th>w/b</th>
<th>HRWR Type</th>
<th>HRWR (fl. oz./cwt binder)</th>
<th>RMA (fl. oz./cwt binder)</th>
<th>Flow Spread (in.)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.200</td>
<td>MasterGlenium 7920</td>
<td>62</td>
<td>0</td>
<td>9.4</td>
<td>Flow above target; reduce HRWR.</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.200</td>
<td>MasterGlenium 7920</td>
<td>52</td>
<td>0</td>
<td>9.6</td>
<td>Flow increased; reduce HRWR.</td>
</tr>
<tr>
<td>3</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.200</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>0</td>
<td>9.7</td>
<td>Flow increased; reduce HRWR.</td>
</tr>
<tr>
<td>4</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.200</td>
<td>MasterGlenium 7920</td>
<td>38</td>
<td>0</td>
<td>9.2</td>
<td>Flow in target range; reduce w/b.</td>
</tr>
<tr>
<td>5</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.190</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>0</td>
<td>8.4</td>
<td>Flow below target; increase w/b.</td>
</tr>
<tr>
<td>6</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.195</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>0</td>
<td>9.1</td>
<td>Flow in target range; evaluate alternative q.</td>
</tr>
<tr>
<td>7</td>
<td>0.23</td>
<td>0.23</td>
<td>0.87</td>
<td>0.195</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>0</td>
<td>9.1</td>
<td>Flow unchanged; return to q = 0.25 and repeat Trial #6.</td>
</tr>
<tr>
<td>8</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.195</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>0</td>
<td>9.3</td>
<td>Similar flow to Trial #6; add RMA.</td>
</tr>
<tr>
<td>9</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.195</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>2</td>
<td>9.4</td>
<td>Flow spread similar, no significant change in surface; increase RMA.</td>
</tr>
<tr>
<td>10</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.195</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>4</td>
<td>9.3</td>
<td>Flow spread similar; skin formation slightly delayed; increase RMA.</td>
</tr>
<tr>
<td>11</td>
<td>0.25</td>
<td>0.25</td>
<td>0.97</td>
<td>0.195</td>
<td>MasterGlenium 7920</td>
<td>43</td>
<td>6</td>
<td>9.1</td>
<td>Flow spread at target; skin formation delayed; select mixture and evaluate strength.</td>
</tr>
</tbody>
</table>
The mixture recommended for in-plant trial production had a w/b of 0.195, and consisted of Type I/II cement, 25 percent silica fume by weight of cement, and fine sand at a sand/cement ratio of 0.97 by weight. The average compressive strength of three 2-inch cubes, cured and tested as described above, was found to be 25,720 psi, which was estimated to correspond to a compressive strength of approximately 23,100 psi for 3-inch by 6-inch cylinders. Therefore, it was predicted that the UHPC mixture designed for Precaster C (Table 2.1.2.3-3) has the potential to achieve a minimum compressive strength of 18,000 psi during field production.

Table 2.1.2.3-3. Recommended Mixture Design for In-Plant Trials - Precaster C

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Amount (lb./yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II Cement (Titan, Patras Greece)</td>
<td>1573</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>BASF MasterLife SF 100</td>
<td>393</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>None</td>
<td>--</td>
</tr>
<tr>
<td>Sand</td>
<td>Mason Sand (ER Jahna, Independent North Mine)</td>
<td>1526</td>
</tr>
<tr>
<td>Water</td>
<td>--</td>
<td>338 (40.5 gal.)</td>
</tr>
<tr>
<td>High-Range Water Reducer</td>
<td>BASF MasterGlenium 7920</td>
<td>59 (845 fl. oz.)</td>
</tr>
<tr>
<td>Rheology Modifying Admixture</td>
<td>GCP V-MAR F100</td>
<td>8 (118 fl. oz.)</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>Steel Fiber (Dura)</td>
<td>263</td>
</tr>
</tbody>
</table>

2.1.2.4 Precaster D

Based on discussions with plant personnel, the candidate materials and preliminary mixture proportions listed in Table 2.1.2.4-1 were selected for initial laboratory trial batching for Precaster D. Preliminary mixture proportions were based on an assumed q value of 0.30. All materials met the recommendations listed above, with the exception that the limestone powder was slightly coarser than the cement. Preliminary mixture development (trial batching) was based on the current HRWR used by Precaster D, RussTech Superflo 2040RM.

Table 2.1.2.4-1. Candidate Mixture - Precaster D

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Relative Proportion, by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II White Cement (Lehigh, Aalborg Plant)</td>
<td>1.00</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>RussTech CSF Condensed Silica Fume</td>
<td>0.20</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>Limestone Powder (Carmeuse Lime &amp; Stone)</td>
<td>0.15</td>
</tr>
<tr>
<td>Sand</td>
<td>#55 sand (Sterling Sand, Lugoff, SC)</td>
<td>1.32</td>
</tr>
<tr>
<td>Error (q = 0.30)</td>
<td>--</td>
<td>177</td>
</tr>
</tbody>
</table>

A series of 21 laboratory trial batches were produced based on the selected candidate mixture design, varying the parameters listed in Table 2.1.1-1 until the minimum w/b and HRWR dosage achieving a flow spread of 9 ± 0.25 inches was identified. A summary of each mixture tested and the adjustments made between batches is presented in Table 2.1.2.4-2. It was observed that by reducing the "q" shape parameter from 0.30 to 0.25, reductions in w/b and increases in silica fume content could be made without negatively reducing the workability of the mixture. The best performance was achieved for a silica-fume only mixture containing the Superflo 2040RM HRWR at a dosage of 34 fl. oz./cwt of binder and with a "q" value of 0.25 (Trial #20).
### Table 2.1.2.4-2. Trial Batch Summary - Precaster D

<table>
<thead>
<tr>
<th>Trial</th>
<th>q</th>
<th>SF/Cement Ratio</th>
<th>LS/Cement Ratio</th>
<th>Sand/Cement Ratio</th>
<th>w/b</th>
<th>HRWR Type</th>
<th>HRWR (fl. oz./cwt binder)</th>
<th>Flow Spread (in.)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.163</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>--</td>
<td>Did not turn fluid; increase w/b</td>
</tr>
<tr>
<td>2</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.170</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>--</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>3</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.177</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>--</td>
<td>Did not flow; increase w/b</td>
</tr>
<tr>
<td>4</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.185</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>--</td>
<td>Did not flow; dry wet sand to near SSD and repeat</td>
</tr>
<tr>
<td>5</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.185</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>5.3</td>
<td>Flow less than target; increase w/b</td>
</tr>
<tr>
<td>6</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.193</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>6.4</td>
<td>Flow less than target; increase w/b</td>
</tr>
<tr>
<td>7</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.200</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>6.6</td>
<td>Flow less than target; increase HRWR</td>
</tr>
<tr>
<td>8</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.200</td>
<td>Superflo 2040RM</td>
<td>83</td>
<td>6.6</td>
<td>Flow did not change; reduce HRWR and increase w/b</td>
</tr>
<tr>
<td>9</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.207</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>7.7</td>
<td>Flow less than target; increase w/b</td>
</tr>
<tr>
<td>10</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.219</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>8.1</td>
<td>Flow less than target; increase w/b</td>
</tr>
<tr>
<td>11</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.230</td>
<td>Superflo 2040RM</td>
<td>63</td>
<td>9.1</td>
<td>Flow at target; reduce HRWR</td>
</tr>
<tr>
<td>12</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.230</td>
<td>Superflo 2040RM</td>
<td>42</td>
<td>9.9</td>
<td>Flow above target; reduce HRWR</td>
</tr>
<tr>
<td>13</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.230</td>
<td>Superflo 2040RM</td>
<td>37</td>
<td>10.1</td>
<td>Flow increased; reduce HRWR</td>
</tr>
<tr>
<td>14</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.230</td>
<td>Superflo 2040RM</td>
<td>31</td>
<td>10.3</td>
<td>Flow increased; reduce w/b</td>
</tr>
<tr>
<td>15</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.222</td>
<td>Superflo 2040RM</td>
<td>31</td>
<td>10.1</td>
<td>Flow still above target; reduce HRWR</td>
</tr>
<tr>
<td>16</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.222</td>
<td>Superflo 2040RM</td>
<td>26</td>
<td>9.9</td>
<td>Flow slightly decreased; reduce w/b</td>
</tr>
<tr>
<td>17</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>1.32</td>
<td>0.215</td>
<td>Superflo 2040RM</td>
<td>32</td>
<td>9.0</td>
<td>Flow at target; evaluate alternative q</td>
</tr>
<tr>
<td>18</td>
<td>0.25</td>
<td>0.25</td>
<td>0.18</td>
<td>1.10</td>
<td>0.202</td>
<td>Superflo 2040RM</td>
<td>29</td>
<td>8.6</td>
<td>Flow near target; evaluate alternative q</td>
</tr>
<tr>
<td>19</td>
<td>0.35</td>
<td>0.19</td>
<td>0.25</td>
<td>1.74</td>
<td>0.201</td>
<td>Superflo 2040RM</td>
<td>31</td>
<td>4.1</td>
<td>Did not flow; re-batch at q = 0.25 with increased HRWR</td>
</tr>
<tr>
<td>20</td>
<td>0.25</td>
<td>0.25</td>
<td>0.18</td>
<td>1.10</td>
<td>0.202</td>
<td>Superflo 2040RM</td>
<td>34</td>
<td>9.1</td>
<td>Flow at target; verify HRWR dosage; evaluate strength</td>
</tr>
<tr>
<td>21</td>
<td>0.25</td>
<td>0.25</td>
<td>0.18</td>
<td>1.10</td>
<td>0.202</td>
<td>Superflo 2040RM</td>
<td>39</td>
<td>8.6</td>
<td>Flow reduced, Trial 20 is at optimal HRWR dosage.</td>
</tr>
</tbody>
</table>
The mixture recommended for in-plant trial production had a w/b of 0.202, and consisted of Type I/II cement, 25 percent silica fume by weight of cement, 18 percent limestone powder by weight of cement, and fine sand at a sand/cement ratio of 1.10 by weight. The average compressive strength of three 2-inch cubes prepared from this mixture (without fibers) and cured and tested as described above, was found to be 23,810 psi, which was estimated to correspond to a compressive strength of approximately 21,400 psi for 3-inch by 6-inch cylinders. Therefore, it was predicted that the UHPC mixture designed for Precaster D (Table 2.1.2.4-3) has the potential to achieve a minimum compressive strength of 18,000 psi during field production.

### Table 2.1.2.4-3. Recommended Mixture Design for In-Plant Trials - Precaster D

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Amount (lb./yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II White Cement (Lehigh, Aalborg Plant)</td>
<td>1,379</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>RussTech CSF Condensed Silica Fume</td>
<td>345</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>Limestone Powder (Carmeuse Lime &amp; Stone)</td>
<td>248</td>
</tr>
<tr>
<td>Sand</td>
<td>#55 sand (Sterling Sand, Lugoff, SC)</td>
<td>1,514</td>
</tr>
<tr>
<td>Water</td>
<td>--</td>
<td>373 (44.7 gal.)</td>
</tr>
<tr>
<td>High-Range Water Reducer</td>
<td>RussTech Superflow 2040RM</td>
<td>48 (671 fl. oz.)</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>Steel Fiber (Dura)</td>
<td>263</td>
</tr>
</tbody>
</table>

**2.1.2.5 Precaster E**

Based on discussions with plant personnel, the candidate materials and preliminary mixture proportions listed in Table 2.1.2.5-1 were selected for initial laboratory trial batching for Precaster E. Preliminary mixture proportions were based on an assumed q value of 0.25. The materials generally met the recommendations listed above, with the exception of the Type I/II cement, which had a Blaine fineness of 432 m²/g, slightly greater than the recommended limit of 400 m²/g, and the #10 sand, which was reported to contain a small amount of potentially deleterious material per ASTM C33. Preliminary mixture development (trial batching) was based on the Chryso Premia 150 HRWR, which was selected by plant personnel based on previous UHPC batching efforts.

### Table 2.1.2.5-1. Candidate Mixture - Precaster E

<table>
<thead>
<tr>
<th>Material</th>
<th>Type/Source</th>
<th>Relative Proportion, by Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II (Ash Grove, Louisville, NE)</td>
<td>1.00</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>Force 10,000D Microsilica (GCP)</td>
<td>0.25</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>Class C Fly Ash (Kansas City Fly Ash, Iatan)</td>
<td>0</td>
</tr>
<tr>
<td>Sand</td>
<td>#10 Sand</td>
<td>1.35</td>
</tr>
<tr>
<td>Error (q = 0.25)</td>
<td>--</td>
<td>154</td>
</tr>
</tbody>
</table>

A series of 17 laboratory trial batches were produced based on the selected candidate mixture design, varying the parameters listed in Table 2.1.1-1 until the minimum w/b and HRWR dosage achieving a flow spread of 9 ± 0.25 inches was identified. A summary of each mixture tested and the adjustments made between batches is presented in Table 2.1.2.5-2. During trial batching, it was observed that dry-mixing was more effective when the sand was in a moisture state near the saturated surface-dry (SSD) condition, rather than wet or oven dry. It was also observed that for the same w/b, using the materials and proportions selected, a greater flow spread was achieved for a mixture containing silica fume only, compared to a mixture consisting of silica fume and fly ash. Therefore, the remaining mixture development was based on a mixture containing silica fume only. The best performance was achieved for a silica-fume only mixture containing the Chryso Premia 150 HRWR at a dosage of 64 fl. oz./cwt of binder, with a “q” value of 0.25 (Trial #16). At the same HRWR dosage, q, and w/b, insufficient flow was achieved using the plant’s current HRWR, GCP Adva Cast 585.
### Table 2.1.2.5-2. Trial Batch Summary - Precaster E

<table>
<thead>
<tr>
<th>Trial</th>
<th>q</th>
<th>SF/Cement Ratio</th>
<th>FA/Cement Ratio</th>
<th>Sand/Cement Ratio</th>
<th>w/b</th>
<th>HRWR Type</th>
<th>HRWR (fl. oz./cwt binder)</th>
<th>Flow Spread (in.)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.192</td>
<td>Premia 150</td>
<td>46</td>
<td>4.0</td>
<td>Did not flow; change q</td>
</tr>
<tr>
<td>2</td>
<td>0.30</td>
<td>0.21</td>
<td>0</td>
<td>1.70</td>
<td>0.198</td>
<td>Premia 150</td>
<td>48</td>
<td>4.0</td>
<td>Did not flow; return to q = 0.25, add fly ash, and increase w/b</td>
</tr>
<tr>
<td>3</td>
<td>0.25</td>
<td>0.25</td>
<td>0.13</td>
<td>1.48</td>
<td>0.216</td>
<td>Premia 150</td>
<td>42</td>
<td>8.8</td>
<td>Flow in target range; repeat trial #2 without fly ash at higher w/b</td>
</tr>
<tr>
<td>4</td>
<td>0.30</td>
<td>0.21</td>
<td>0</td>
<td>1.70</td>
<td>0.223</td>
<td>Premia 150</td>
<td>48</td>
<td>8.6</td>
<td>Flow less than Trial #3; repeat trial #1 without fly ash at higher w/b</td>
</tr>
<tr>
<td>5</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.216</td>
<td>Premia 150</td>
<td>46</td>
<td>10.7</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>6</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.208</td>
<td>Premia 150</td>
<td>46</td>
<td>9.7</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>7</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.192</td>
<td>Premia 150</td>
<td>46</td>
<td>4.9</td>
<td>Flow below target; increase w/b</td>
</tr>
<tr>
<td>8</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.200</td>
<td>Premia 150</td>
<td>46</td>
<td>8.3</td>
<td>Flow below target; increase w/b</td>
</tr>
<tr>
<td>9</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.204</td>
<td>Premia 150</td>
<td>46</td>
<td>9.1</td>
<td>Flow at target; increase HRWR</td>
</tr>
<tr>
<td>10</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.200</td>
<td>Premia 150</td>
<td>51.8</td>
<td>9.5</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>11</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.196</td>
<td>Premia 150</td>
<td>51.8</td>
<td>8.9</td>
<td>Flow at target; increase HRWR</td>
</tr>
<tr>
<td>12</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.196</td>
<td>Premia 150</td>
<td>57.5</td>
<td>9.6</td>
<td>Flow above target; decrease w/b</td>
</tr>
<tr>
<td>13</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.192</td>
<td>Premia 150</td>
<td>57.5</td>
<td>8.9</td>
<td>Flow at target; increase HRWR</td>
</tr>
<tr>
<td>14</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.192</td>
<td>Premia 150</td>
<td>64</td>
<td>9.2</td>
<td>Flow near target; increase HRWR</td>
</tr>
<tr>
<td>15</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.192</td>
<td>Premia 150</td>
<td>69</td>
<td>9.2</td>
<td>Flow unchanged; decrease w/b</td>
</tr>
<tr>
<td>16</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.188</td>
<td>Premia 150</td>
<td>64</td>
<td>8.9</td>
<td>Flow at target; change HRWR; evaluate strength</td>
</tr>
<tr>
<td>17</td>
<td>0.25</td>
<td>0.25</td>
<td>0</td>
<td>1.35</td>
<td>0.189</td>
<td>Adva Cast 585</td>
<td>64</td>
<td>4.0</td>
<td>Did not flow; Trial #16 is at optimal proportions.</td>
</tr>
</tbody>
</table>
The mixture recommended for in-plant trial production had a w/b of 0.188, and consisted of Type I/II cement, 25 percent silica fume by weight of cement, and fine sand at a sand/cement ratio of 1.35 by weight. The average compressive strength of three 2-inch cubes prepared from this mixture (without fibers) and cured and tested as described above, was found to be 26,270 psi, which was estimated to correspond to a compressive strength of approximately 23,600 psi for 3-inch by 6-inch cylinders. Therefore, it was predicted that the UHPC mixture designed for Precaster E (Table 2.1.2.5-3) has the potential to achieve a minimum compressive strength of 18,000 psi during field production.

2.1.2.6 Summary
All final mixtures demonstrated satisfactory workability and compressive strength when tested as described above. The final mixtures recommended to the precasters for plant-scale trial implementation are listed in Table 2.1.2.6-1. These mixture proportions do not consider air content in the 1 cubic-yard estimated yield. The mixtures represent three different classes of cement (Type I/II, Type III, and White Type I/II), three different HRWR types (Chryso Premia 150, BASF MasterGlenium 7920, and RussTech Superflo 2040RM), and two different supplemental materials (silica fume only [no supplemental material] and silica fume + limestone). It was noted that, due to the greater efficiency of batch plant mixers, reductions in the required HRWR dosage may be expected when the mixtures are scaled up to production batch size.

Table 2.1.2.6-1. Summary of UHPC Mixtures - Amount per yd³

<table>
<thead>
<tr>
<th>Material</th>
<th>Precaster A</th>
<th>Precaster B</th>
<th>Precaster C</th>
<th>Precaster D</th>
<th>Precaster E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>1,514 lb. (Type III)</td>
<td>1,297 lb. (Type I/II)</td>
<td>1,573 lb. (Type I/II)</td>
<td>1,379 lb. (White Type I/II)</td>
<td>1,404 lb. (Type I/II)</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>378 lb.</td>
<td>324 lb.</td>
<td>393 lb.</td>
<td>345 lb.</td>
<td>351 lb.</td>
</tr>
<tr>
<td>Supplemental Material</td>
<td>--</td>
<td>143 lb. (Limestone)</td>
<td>--</td>
<td>248 lb. (Limestone)</td>
<td>--</td>
</tr>
<tr>
<td>Sand</td>
<td>1,665 lb.</td>
<td>1,881 lb.</td>
<td>1,526 lb.</td>
<td>1,514 lb.</td>
<td>1,896 lb.</td>
</tr>
<tr>
<td>Water</td>
<td>304 lb. (36.4 gal.)</td>
<td>288 lb. (34.5 gal.)</td>
<td>338 lb. (40.5 gal.)</td>
<td>373 lb. (44.7 gal.)</td>
<td>275 lb. (32.9 gal.)</td>
</tr>
<tr>
<td>High-Range Water Reducer</td>
<td>105 lb. (1523 fl. oz.)</td>
<td>57 lb. (811 fl. oz.)</td>
<td>59 lb. (845 fl. oz.)</td>
<td>48 lb. (671 fl. oz.)</td>
<td>78 lb. (1123 fl. oz.)</td>
</tr>
<tr>
<td>Other Admixture</td>
<td>--</td>
<td>--</td>
<td>8 lb. (118 fl. oz.)</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>263 lb.</td>
<td>263 lb.</td>
<td>263 lb.</td>
<td>263 lb.</td>
<td>263 lb.</td>
</tr>
<tr>
<td>w/b</td>
<td>0.200</td>
<td>0.184</td>
<td>0.195</td>
<td>0.202</td>
<td>0.188</td>
</tr>
<tr>
<td>Flow Spread</td>
<td>8.6 in.</td>
<td>9.2 in.</td>
<td>9.1 in.</td>
<td>9.1 in.</td>
<td>8.9 in.</td>
</tr>
<tr>
<td>Compressive Strength (2-inch cubes)</td>
<td>27,820 psi</td>
<td>26,020 psi</td>
<td>25,720 psi</td>
<td>23,810 psi</td>
<td>26,270 psi</td>
</tr>
</tbody>
</table>
2.2 UHPC Mixture In-Plant Trial Batching

In-plant production trials were performed for the six participating precasters in Spring and Summer 2019. The primary objectives of the in-plant trial batches were to:

- Verify that UHPC could be produced and placed using each plant's production equipment;
- Provide assistance to plant personnel for batching, placing, and finishing UHPC in accordance with the Guidelines for Production of UHPC developed for this project;
- Train quality control personnel to conduct fresh property tests (i.e., flow spread) and to fabricate test specimens for performance evaluations in accordance with appropriate industry standards (e.g., ASTM C1856 Standard Practice for Fabricating and Testing Specimens of Ultra-High-Performance Concrete); and
- Observe the characteristics of the as-batched UHPC as it is placed into mock-up element forms and finished.

Prior to each in-plant trial, a pre-trial meeting was held with eConstruct, WJE, and plant personnel to discuss the production process and to plan for the production trials. Discussion topics included:

- Batching and Mixing
- Material Transport and Placement
- Workability Retention for Large Pours
- Finishing and Curing

While the precasters were provided the recommended mixtures, they were encouraged to produce trial batches on their own and to modify the mixture proportions or adopt alternate mixture designs based on other material sources if they felt the alternatives were more compatible with their production operations. Each in-plant trial was conducted over a two-day period. The first day generally consisted of the pre-trial meeting and typically one or two trial batches to fine-tune the dosages of chemical admixtures necessary to achieve the target workability characteristics for the fresh UHPC, and the second day generally consisted of mock-up element fabrication.

Tabulated summaries and photographs of each in-plant trial are presented in the following sections. The as-batched mixture design and the mixing sequence (including timing) presented for each plant reflect the mixture that was used to produce test specimens for laboratory characterization of the UHPC material performance. This was typically the last batch (or batches) produced at each plant. These mixture proportions and batching sequences were replicated in the laboratory for the shrinkage evaluations discussed in Section 2.3.2.

2.2.1 Precaster A

Date of Production Trial: May 1-2, 2019
Weather Conditions: 50 - 60 °F, cloudy with light breeze

2.2.1.1 Batching

<table>
<thead>
<tr>
<th>Mixer Type</th>
<th>Planetary pan mixer (Figure 2.2.1.7-1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixer Capacity</td>
<td>3 cubic yards</td>
</tr>
<tr>
<td>Batch Volume</td>
<td>1 cubic yard</td>
</tr>
<tr>
<td>Number of Batches Produced</td>
<td>2: 1 trial batch on 5/1, 1 trial batch on 5/2</td>
</tr>
</tbody>
</table>

**Batching Notes**
- Material was batched in a mixer that had been “buttered” with conventional concrete.
- Aggregate moisture content was measured by burn-off. Then aggregates were pre-weighed in supersacks for 1 cubic yard of material. Batch water was adjusted manually based on measured moisture content.
- Fibers were added by shaking over a screen. (Figure 2.2.1.7-2)

2.2.1.2 Mixture Design (As Batched)

<table>
<thead>
<tr>
<th>Material</th>
<th>Batch #52-1 (per cubic yard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume Batched</td>
<td>1.0 yd³</td>
</tr>
<tr>
<td>Type III Cement</td>
<td>1,500 lb.</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>375 lb.</td>
</tr>
<tr>
<td>Sand</td>
<td>1,655 lb.</td>
</tr>
<tr>
<td>Water</td>
<td>277 lb.</td>
</tr>
</tbody>
</table>
2.2.1.3 Batching Sequence

<table>
<thead>
<tr>
<th>Step</th>
<th>Approximate Duration, min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Add sand and silica fume, mix 5 min.</td>
<td>9</td>
</tr>
<tr>
<td>2. Add cement, mix 1 min.</td>
<td>3.5</td>
</tr>
<tr>
<td>3. Add water, mix 5 min.</td>
<td>6</td>
</tr>
<tr>
<td>4. Add HRWR.</td>
<td>2.5</td>
</tr>
<tr>
<td>5. Mix until turn.</td>
<td>8.5</td>
</tr>
<tr>
<td>6. Mix 6 min., then sample and measure flow spread.</td>
<td>8</td>
</tr>
<tr>
<td>7. Add fibers.</td>
<td>13.5 (~3 min. after flow test)</td>
</tr>
<tr>
<td>8. Mix 2 min., then sample, measure flow spread, and discharge.</td>
<td>3.5</td>
</tr>
<tr>
<td>Total mixing time</td>
<td>57 min.</td>
</tr>
</tbody>
</table>

2.2.1.4 Fresh UHPC Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Target</th>
<th>Batch 52-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature at discharge</td>
<td>65 to 85 °F</td>
<td>63 °F</td>
</tr>
<tr>
<td>Temperature at point of placement</td>
<td>65 to 85 °F</td>
<td>59 °F</td>
</tr>
<tr>
<td>Flow spread before fiber addition</td>
<td>8 to 10 in.</td>
<td>8 3/4 in.</td>
</tr>
<tr>
<td>Flow spread at discharge</td>
<td>8 to 10 in.</td>
<td>9 3/8 in.</td>
</tr>
<tr>
<td>Flow spread at point of placement</td>
<td>8 to 10 in.</td>
<td>10 1/4 in.</td>
</tr>
<tr>
<td>Unit weight</td>
<td>152.7 lb/ft³ (Assuming no air)</td>
<td>148.8 lb/ft³ [3.3% air]</td>
</tr>
</tbody>
</table>

**Fresh UHPC Observations**

- Mixing was comparatively slower than for other plants due to relatively large diameter of pan mixer (less shearing action).
- Additional mixing time improved workability.

2.2.1.5 Element Production

<table>
<thead>
<tr>
<th>Element(s) Produced</th>
<th>4 panels for stay-in-place deck panel forms (Figure 2.2.1.7-3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method for Transporting Material to Forms</td>
<td>Auger-fed delivery truck (Figure 2.2.1.7-4)</td>
</tr>
<tr>
<td>Consolidation</td>
<td>None</td>
</tr>
<tr>
<td>Finishing</td>
<td>Screed and roller (Figures 2.2.1.7-5 and 2.2.1.7-6)</td>
</tr>
<tr>
<td>Curing</td>
<td>Applied evaporation reducer to surface immediately after finishing, covered with plastic sheet and applied heat under tent</td>
</tr>
<tr>
<td>Thermal Treatment</td>
<td>None</td>
</tr>
</tbody>
</table>

**Production Notes**

- To avoid elephant skin, first 2 panels were placed in less than 10 min., struck off immediately and finished with a spiked roller, then sprayed with evaporation reducer.
- Second 2 panels were finished 50 min. after initial discharge from mixer. Elephant skin first started to form around this time.
2.2.1.6 Hardened UHPC Properties

| Test Specimens Produced                                                                 | 6 3x6 cylinders cured in lab |
|                                                                                     | 6 3x6 cylinders cured with product |
|                                                                                     | 3 4x4x14 beams cured in lab |
|                                                                                     | 3 4x4x14 beams cured with product |
|                                                                                     | 4 stressed and 4 unstressed strand bond samples with 4x8 inch cross-section, 6, 12, 18, and 24 inches in length, and containing three 0.6-inch strands spaced at 2 inches at mid-height |
| 28-day compressive strength (ASTM C1856)                                           | Specimens cured with product (average of 3): 22,840 psi |
|                                                                                     | Lab-cured specimens (average of 3): 24,190 psi |
| 28-day flexural strength (ASTM C1856)                                              | Lab-cured specimens (average of 3): |
|                                                                                     | First-peak flexural strength, \( f_{fc} \): 2,360 psi |
|                                                                                     | Peak flexural strength, \( f_{fu} \): 2,470 psi (105% of \( f_{fc} \)) |
|                                                                                     | Residual flexural strength at L/300: 91% of \( f_{fc} \) |
|                                                                                     | Residual flexural strength at L/150: 78% of \( f_{fc} \) |

2.2.1.7 Photographs - Precaster A

![Figure 2.2.1.7-1. Planetary Pan Mixer (Precaster A)](image1)  
![Figure 2.2.1.7-2. Addition of Fibers to the Mixer (Precaster A)](image2)
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Precaster Mix Design Characterization

Figure 2.2.1.7-3. Panel Forms (Precaster A)

Figure 2.2.1.7-4. Filling Forms using an Auger-Fed Delivery Truck (Precaster A)

Figure 2.2.1.7-5. Screeding the Surface (Precaster A)

Figure 2.2.1.7-6. Finishing the Surface with a Spiked Roller (Precaster A)
2.2.2 Precaster B

Date of Production Trial | April 22-23, 2019
Weather Conditions | 55 – 75 °F, sunny with light breeze

2.2.2.1 Batching

| Mixer Type | Planetary pan mixer (Figure 2.2.2.7-1) |
| Mixer Capacity | 5 cubic yards |
| Batch Volume | 2-3 cubic yard |
| Number of Batches Produced | 2 |

**Batching Notes**

- Batch #1 was prepared in a clean, dry mixer. Batch #2 was produced in a “buttered” mixer.
- Aggregate moisture content was measured by burn-off, with the aggregate sampled after charging into the mixer. Batch water was adjusted based on measured moisture content.
- Silica fume, limestone powder, and fibers were added manually from bags (Figure 2.2.2.7-2).
- The mixture design developed in the laboratory was based on a Type I/II cement, but a Type III cement was used for production trial.
- A hydration stabilizer (MasterSure Delvo) was added to the mixture to extend working time.

2.2.2.2 Mixture Design (As Batched)

<table>
<thead>
<tr>
<th>Material</th>
<th>Batch #2 (per cubic yard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume Batched</td>
<td>3.0 yd³</td>
</tr>
<tr>
<td>Type III Cement</td>
<td>1,272 lb.</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>318 lb.</td>
</tr>
<tr>
<td>Limestone Powder</td>
<td>140 lb.</td>
</tr>
<tr>
<td>Sand</td>
<td>1,848 lb.</td>
</tr>
<tr>
<td>Water</td>
<td>269 lb.</td>
</tr>
<tr>
<td>High-Range Water Reducer (HRWR) - 36% solids</td>
<td>1,001 fl. oz.</td>
</tr>
<tr>
<td>Hydration Stabilizer - 20% solids</td>
<td>86 fl. oz.</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>262 lb.</td>
</tr>
<tr>
<td>w/b</td>
<td>0.184</td>
</tr>
<tr>
<td>w/c</td>
<td>0.250</td>
</tr>
<tr>
<td>s/c</td>
<td>1.51</td>
</tr>
<tr>
<td>% silica fume by wt. cement</td>
<td>25%</td>
</tr>
<tr>
<td>% limestone powder by wt. cement</td>
<td>11%</td>
</tr>
<tr>
<td>HRWR dosage, fl. oz./cwt binder</td>
<td>57.9</td>
</tr>
<tr>
<td>Hydration stabilizer dosage, fl. oz./cwt binder</td>
<td>5.0</td>
</tr>
</tbody>
</table>

2.2.2.3 Batching Sequence

<table>
<thead>
<tr>
<th>Step</th>
<th>Approximate Duration, min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Add all dry materials to mixer.</td>
<td>8.5</td>
</tr>
<tr>
<td>2. Dry mix for 3 min.</td>
<td>3</td>
</tr>
<tr>
<td>3. Add hydration stabilizer and mix for 1 min.</td>
<td>1</td>
</tr>
<tr>
<td>4. Add mixing water and mix for 1 min.</td>
<td>1.5</td>
</tr>
<tr>
<td>5. Add HRWR and mix until turn.</td>
<td>5</td>
</tr>
<tr>
<td>6. Mix 2 min., then sample and measure flow spread.</td>
<td>6.5</td>
</tr>
<tr>
<td>7. Add fibers.</td>
<td>19</td>
</tr>
<tr>
<td>8. Mix 2 min., then sample, measure flow spread, and discharge.</td>
<td>2.5</td>
</tr>
<tr>
<td>Total mixing time</td>
<td>47 min.</td>
</tr>
</tbody>
</table>
2.2.2.4 Fresh UHPC Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Target</th>
<th>Batch #2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature at discharge</td>
<td>65 to 85 °F</td>
<td>81 °F</td>
</tr>
<tr>
<td>Temperature at point of placement</td>
<td>65 to 85 °F</td>
<td>Not recorded</td>
</tr>
<tr>
<td>Flow spread before fiber addition</td>
<td>8 to 10 in.</td>
<td>8 1/4 in.</td>
</tr>
<tr>
<td>Flow spread at discharge</td>
<td>8 to 10 in.</td>
<td>8 3/4 in.</td>
</tr>
<tr>
<td>Flow spread at point of placement</td>
<td>8 to 10 in.</td>
<td>8 1/4 in.</td>
</tr>
<tr>
<td>Unit weight</td>
<td>155.1 lb/ft³</td>
<td>150.7 lb/ft³ [3.4% air]</td>
</tr>
</tbody>
</table>

Fresh UHPC Observations
- Additional HRWR was added to both batches prior to fiber addition to help improve initial workability.
- Fibers segregated and clumped in Batch #1 to extent that material could not be placed from truck chute (Figure 2.2.7.3).
- Batch #2 also exhibited some clumping, but fibers were added more gradually and mix consistency was generally better.
- Flow spread for Batch #2 remained above 7 inches for approximately 45 minutes after discharging from mixer.

2.2.2.5 Element Production

<table>
<thead>
<tr>
<th>Element(s) Produced</th>
<th>Voided box slabs (Figures 2.2.7.4 and 2.2.7.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method for Transporting Material to Forms</td>
<td>Discharge into mixing truck, place from bucket into forms (Figure 2.2.7.6)</td>
</tr>
<tr>
<td>Consolidation</td>
<td>External vibration</td>
</tr>
<tr>
<td>Finishing</td>
<td>N/A</td>
</tr>
<tr>
<td>Curing</td>
<td>N/A</td>
</tr>
<tr>
<td>Thermal Treatment</td>
<td>None</td>
</tr>
</tbody>
</table>

Production Notes
- Slab was attempted in two lifts: The base of box slab was placed first, then void former was inserted and additional material was placed on top until the forms were filled. This was not successful and placement was abandoned.
- Significant fiber clumping was observed, especially for Batch #1.

2.2.2.6 Hardened UHPC Properties

<table>
<thead>
<tr>
<th>Test Specimens Produced</th>
<th>6 4x8 cylinders cured in lab</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6 4x8 cylinders match-cured with product</td>
</tr>
<tr>
<td></td>
<td>3 4x4x14 beams cured on bed with product</td>
</tr>
<tr>
<td></td>
<td>4 stressed and 4 unstressed strand bond samples with 3x6 inch cross-section, 6, 12, 18, and 24 inches in length, and containing three 0.5-inch special strands spaced at 2 inches at mid-height</td>
</tr>
</tbody>
</table>

28-day compressive strength (ASTM C1856, performed on 4x8 cylinders)
- Match-cured specimens (average of 3): 17,830 psi
- Lab-cured specimens (average of 3): 17,900 psi

28-day flexural strength (ASTM C1856)
- Lab-cured specimens (average of 3):
  - First-peak flexural strength, \( f_{fc} \): 2,190 psi
  - Peak flexural strength, \( f_{fu} \): 3,620 psi (166% of \( f_{fc} \))
  - Residual flexural strength at L/300: 161% of \( f_{fc} \)
  - Residual flexural strength at L/150: 129% of \( f_{fc} \)
2.2.2.7 Photographs - Precaster B

*Figure 2.2.2.7-1. Planetary Pan Mixer (Precaster B)*

*Figure 2.2.2.7-2. Adding Fibers to the Mixer (Precaster B)*
Figure 2.2.2.7-3. Fiber Clumps (Precaster B)

Figure 2.2.2.7-4. Box Slab Forms (Precaster B)

Figure 2.2.2.7-5. Material Placed at the Base of the Box Slab Form (Precaster B)

Figure 2.2.2.7-6. Discharging UHPC from the Truck into a Bucket for Placement (Precaster B)
2.2.3 Precaster C

<table>
<thead>
<tr>
<th>Date of Production Trial</th>
<th>April 17-18, 2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weather Conditions</td>
<td>87 °F, sunny with a light breeze</td>
</tr>
</tbody>
</table>

2.2.3.1 Batching

<table>
<thead>
<tr>
<th>Mixer Type</th>
<th>Horizontal Twin Shaft (Figure 2.2.3.7-1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixer Capacity</td>
<td>6 cubic yards</td>
</tr>
<tr>
<td>Batch Volume</td>
<td>3.15 cubic yards</td>
</tr>
<tr>
<td>Number of Batches Produced</td>
<td>4: 2 batches without fibers on 4/17, 2 batches with fibers on 4/18</td>
</tr>
</tbody>
</table>

**Batching Notes**
- Element required 5.6 yd³ for production; two 3.15 yd³ batches were produced sequentially and combined in a mixing truck prior to placement.
- Batch #1 was mixed in a clean, surface-saturated mixer, then discharged into mixing truck. Batch #2 was mixed immediately afterwards in the uncleaned mixer, then discharged into mixing truck.
- Aggregate moisture content was measured by moisture probe and water content was adjusted automatically by batch plant computer.
- Rheology modifying admixture and steel fibers were weighed and added manually (Figure 2.2.3.7-2); all other materials were weighed and added by batch plant.

2.2.3.2 Mixture Design (As Batched)

<table>
<thead>
<tr>
<th>Material</th>
<th>Batch #1 (per cubic yard)</th>
<th>Batch #2 (per cubic yard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume Batched</td>
<td>3.13 yd³</td>
<td>3.15 yd³</td>
</tr>
<tr>
<td>Type I/II Cement</td>
<td>1,570 lb.</td>
<td>1,565 lb.</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>398 lb.</td>
<td>394 lb.</td>
</tr>
<tr>
<td>Sand</td>
<td>1,525 lb.</td>
<td>1,534 lb.</td>
</tr>
<tr>
<td>Water</td>
<td>339 lb.</td>
<td>341 lb.</td>
</tr>
<tr>
<td>High-Range Water Reducer (HRWR) - 36% solids</td>
<td>849 fl. oz.</td>
<td>757 fl. oz.</td>
</tr>
<tr>
<td>Rheology Modifying Admixture (RMA) - 3.5% solids</td>
<td>119 fl. oz.</td>
<td>118 fl. oz.</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>265 lb.</td>
<td>263 lb.</td>
</tr>
<tr>
<td>w/b</td>
<td>0.196</td>
<td>0.195</td>
</tr>
<tr>
<td>w/c</td>
<td>0.245</td>
<td>0.244</td>
</tr>
<tr>
<td>s/c</td>
<td>0.97</td>
<td>0.98</td>
</tr>
<tr>
<td>% silica fume by wt. cement</td>
<td>25%</td>
<td>25%</td>
</tr>
<tr>
<td>HRWR dosage, fl. oz./cwt binder</td>
<td>43.1</td>
<td>38.6</td>
</tr>
<tr>
<td>RMA dosage, fl. oz./cwt binder</td>
<td>6.0</td>
<td>6.0</td>
</tr>
</tbody>
</table>

2.2.3.3 Batching Sequence

<table>
<thead>
<tr>
<th>Step</th>
<th>Approximate Duration, min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Add sand to mixer.</td>
<td>0.5 - 1</td>
</tr>
<tr>
<td>2. Add silica fume to mixer, mix 1 min.</td>
<td>3.5 - 4.5</td>
</tr>
<tr>
<td>3. Add 30% of mixing water + 30% of HRWR, mix for 1 min.</td>
<td>2.5 - 3</td>
</tr>
<tr>
<td>4. Add cement, mix for 1 min.</td>
<td>2.5</td>
</tr>
<tr>
<td>5. Add remaining mixing water + RMA, mix for 1 min.</td>
<td>2 - 2.5</td>
</tr>
<tr>
<td>6. Add remaining HRWR, mix until turn.</td>
<td>3 - 3.5</td>
</tr>
<tr>
<td>7. Mix 2 min., then sample and measure flow spread.</td>
<td>2</td>
</tr>
<tr>
<td>8. Add fibers.</td>
<td>6 - 9</td>
</tr>
<tr>
<td>9. Mix for 1 min., then sample, measure flow spread, and discharge.</td>
<td>1</td>
</tr>
<tr>
<td>Total mixing time</td>
<td>28 min. (#1), 23 min. (#2)</td>
</tr>
</tbody>
</table>
2.2.3.4 Fresh UHPC Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Target</th>
<th>Batch #1</th>
<th>Batch #2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature at discharge</td>
<td>65 to 85 °F</td>
<td>86 °F</td>
<td>91 °F</td>
</tr>
<tr>
<td>Temperature at point of placement</td>
<td>65 to 85 °F</td>
<td>--</td>
<td>90 °F</td>
</tr>
<tr>
<td>Flow spread before fiber addition</td>
<td>8 to 10 in.</td>
<td>11 7/8 in.</td>
<td>10 1/2 in.</td>
</tr>
<tr>
<td>Flow spread at discharge</td>
<td>8 to 10 in.</td>
<td>12 in.</td>
<td>10 1/4 in.</td>
</tr>
<tr>
<td>Flow spread at point of placement</td>
<td>8 to 10 in.</td>
<td>--</td>
<td>10 3/4 in.</td>
</tr>
<tr>
<td>Unit weight</td>
<td>154.0 lb/ft³ (Assuming no air)</td>
<td>--</td>
<td>149.6 lb/ft³ [2.9% air]</td>
</tr>
</tbody>
</table>

- Fiber clumps segregated during flow spread tests for both batches, especially Batch #1. HRWR reduced by 10% for Batch #2 to reduce flow spread nearer to target.
- Fiber clumping likely due to insufficient separation of fibers as they were dumped from bags.

2.2.3.5 Element Production

<table>
<thead>
<tr>
<th>Element(s) Produced</th>
<th>Box slab beam (Figure 2.2.3.7-3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method for Transporting Material to Forms</td>
<td>Transported via mixing truck, placed into forms via chute behind leading edge (Figures 2.2.3.7-4 and 2.2.3.7-5)</td>
</tr>
<tr>
<td>Consolidation</td>
<td>Slight external vibration</td>
</tr>
<tr>
<td>Finishing</td>
<td>None</td>
</tr>
<tr>
<td>Curing</td>
<td>Tarp (Figure 2.2.3.7-6)</td>
</tr>
<tr>
<td>Thermal Treatment</td>
<td>None</td>
</tr>
</tbody>
</table>

- Concrete readily flowed down sides of forms but flowed only slowly along underside of void formers. Material was placed from center of form, pausing at holes in center of void formers and diaphragms until form was filled.
- Fiber balling and fiber segregation were observed in the chute. Some cement and silica fume balls were also present.

2.2.3.6 Hardened UHPC Properties

- 6 4x8 cylinders cured in lab
- 6 4x8 cylinders cured with product
- 3 4x4x14 beams cured in lab
- 3 4x4x14 beams cured with product
- 4 stressed and 4 unstressed strand bond samples with 6x6 square cross-section, 6, 12, 18, and 24 inches in length, and containing three 0.5-inch strands spaced at 2 inches at mid-height

28-day compressive strength (ASTM C1856, performed on 4x8 cylinders)

- Match-cured specimens (average of 3): 19,780 psi
- Lab-cured specimens (average of 3): 18,970 psi

28-day flexural strength (ASTM C1856)

- Lab-cured specimens (average of 3):
  - First-peak flexural strength, $f_{f1}$: 1,960 psi
  - Peak flexural strength, $f_{fc}$: 3,170 psi (162% of $f_{fc}$)
  - Residual flexural strength at L/300: 160% of $f_{fc}$
  - Residual flexural strength at L/150: 137% of $f_{fc}$
2.2.3.7 Photographs - Precaster C

*Figure 2.2.3.7-1. Horizontal Twin-Shaft Mixer (Precaster C)*

*Figure 2.2.3.7-2. Adding Fibers to the Mixer (Precaster C)*

*Figure 2.2.3.7-3. Box Slab Beam Form (Precaster C)*

*Figure 2.2.3.7-4. Placement into the Form (Precaster C)*
Figure 2.2.3.7-5. Filled UHPC Form (Precaster C)

Figure 2.2.3.7-6. Covering with a Tarp to Cure (Precaster C)
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Precaster Mix Design Characterization

2.2.4 Precaster D

<table>
<thead>
<tr>
<th>Date of Production Trial</th>
<th>April 24-25, 2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weather Conditions</td>
<td>65 - 85 °F, sunny, no wind</td>
</tr>
</tbody>
</table>

2.2.4.1 Batching

<table>
<thead>
<tr>
<th>Mixer Type</th>
<th>Planetary Pan Mixer (Figure 2.2.4.7-1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixer Capacity</td>
<td>2 cubic yards</td>
</tr>
<tr>
<td>Batch Volume</td>
<td>0.5 to 1.5 cubic yards</td>
</tr>
</tbody>
</table>

| Number of Batches Produced  | 7: 4 trial batches produced 4/24 for mix refinement and laboratory testing 3 trial batches produced 4/25 for element fabrication |

**Batching Notes**
- Material was batched in a “buttered” mixer.
- Aggregate moisture content was measured by burn-off, with the aggregate sampled after charging into the mixer. Batch water was adjusted based on measured moisture content.
- Silica fume and limestone powder were added by bag.
- Fibers added through sieve, using manual and mechanical vibration (Figure 2.2.4.7-2).
- A workability-retaining admixture and air detrainer were added during trial batching.
- Only 1.5% fiber volume was used for batches produced on 4/25 due to limited availability of material. However, batch used to produce specimens for hardened concrete testing included 2% fibers by volume.

2.2.4.2 Mixture Design (As Batched)

<table>
<thead>
<tr>
<th>Material</th>
<th>Batch #424-Lab Trial #2 (per cubic yard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume Batched</td>
<td>0.5 yd³</td>
</tr>
<tr>
<td>White Cement</td>
<td>1,380 lb.</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>300 lb.</td>
</tr>
<tr>
<td>Limestone Powder</td>
<td>250 lb.</td>
</tr>
<tr>
<td>Sand</td>
<td>1,522 lb.</td>
</tr>
<tr>
<td>Water</td>
<td>361 lb.</td>
</tr>
<tr>
<td>High-Range Water Reducer (HRWR) - 38% solids</td>
<td>340 fl. oz.</td>
</tr>
<tr>
<td>Workability-Retaining Admixture - 28% solids</td>
<td>380 fl. oz.</td>
</tr>
<tr>
<td>Air Detrainer</td>
<td>2 fl. oz.</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>264 lb.</td>
</tr>
<tr>
<td>w/b</td>
<td>0.205</td>
</tr>
<tr>
<td>w/c</td>
<td>0.286</td>
</tr>
<tr>
<td>s/c</td>
<td>1.10</td>
</tr>
<tr>
<td>% silica fume by wt. cement</td>
<td>22%</td>
</tr>
<tr>
<td>% limestone powder by wt. cement</td>
<td>18%</td>
</tr>
<tr>
<td>HRWR dosage, fl. oz./cwt binder</td>
<td>17.6</td>
</tr>
<tr>
<td>Workability-retaining admixture dosage, fl. oz./cwt binder</td>
<td>19.7</td>
</tr>
</tbody>
</table>

2.2.4.3 Batching Sequence

<table>
<thead>
<tr>
<th>Step</th>
<th>Approximate Duration, min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Add sand and mix.</td>
<td>3</td>
</tr>
<tr>
<td>2. Pre-wet sand with water and air detrainer.</td>
<td>1.5</td>
</tr>
<tr>
<td>3. Add silica fume and mix.</td>
<td>3</td>
</tr>
<tr>
<td>4. Add fibers to dry-mix.</td>
<td>16.5</td>
</tr>
<tr>
<td>5. Add limestone powder.</td>
<td>3</td>
</tr>
<tr>
<td>6. Add cement.</td>
<td>2</td>
</tr>
<tr>
<td>7. Add remaining water.</td>
<td>2.5</td>
</tr>
</tbody>
</table>
8. Add HRWR and workability retaining admixture.  | Precaster Mix Design Characterization | 1.5
9. Add tail water* and mix to turn.  | 3
10. Mix 3 min, then sample, measure flow spread, and discharge.  | 9
Total mixing time  | 44 min.

*Tail water not added to every batch.

### 2.2.4.4 Fresh UHPC Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Target</th>
<th>Batch 424-Lab Trial #2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature at discharge</td>
<td>65 to 85 °F</td>
<td>83 °F</td>
</tr>
<tr>
<td>Temperature at point of placement</td>
<td>65 to 85 °F</td>
<td>88 °F</td>
</tr>
<tr>
<td>Flow spread before fiber addition</td>
<td>8 to 10 in.</td>
<td>N/A</td>
</tr>
<tr>
<td>Flow spread at discharge</td>
<td>8 to 10 in.</td>
<td>10 3/4 in.</td>
</tr>
<tr>
<td>Flow spread at point of placement</td>
<td>8 to 10 in.</td>
<td>9 3/4 in.</td>
</tr>
<tr>
<td>Unit weight</td>
<td>151.7 lb/ft³ (Assuming no air)</td>
<td>147.3 lb/ft³ [3% air]</td>
</tr>
</tbody>
</table>

**Fresh UHPC Observations**
- Fiber balls were observed after approximately 75% of fibers had been added to the mixer.
- Flow spread remained above 7" for approximately 50 minutes after discharge.

### 2.2.4.5 Element Production

**Element(s) Produced**

60-foot joist and deck panels (Figure 2.2.4.7-3)

**Method for Transporting Material to Forms**
- Initial trials on 4/24 used bucket with spout and gate valve, but fibers clogged valve (Figure 2.2.4.7-4).
- Batches on 4/25 were placed from auger-fed delivery truck (Figure 2.2.4.7-5).

**Consolidation**
External and internal vibration

**Finishing**
Added thin layer of water to surface to facilitate finishing

**Curing**
Under plastic

**Thermal Treatment**
None

**Production Notes**
Placement from a single location was challenging. Most successful placement occurred when material was deposited into forms in a back-and-forth motion.

### 2.2.4.6 Hardened UHPC Properties

**Test Specimens Produced**
- 6 3x6 cylinders cured in lab
- 6 3x6 cylinders cured on bed with product
- 3 4x4x14 beams cured in lab
- 3 4x4x14 beams cured on bed with product
- 4 unstressed strand bond samples with 4x8 inch cross-section, 6, 12, 18, and 24 inches in length, and containing three 0.5-inch special strands spaced at 2 inches at mid-height

**28-day compressive strength (ASTM C1856, performed on 3x6 cylinders)**
Specimens cured with product (average of 3): 16,660 psi
Lab-cured specimens (average of 3): 16,290 psi

**28-day flexural strength (ASTM C1856)**
Lab-cured specimens (average of 3):
- First-peak flexural strength, \( f_{pu} \): 2,100 psi
- Peak flexural strength, \( f_{pu} \): 3,250 psi (154% of \( f_{pu} \))
- Residual flexural strength at L/300: 150% of \( f_{pu} \)
- Residual flexural strength at L/150: 111% of \( f_{pu} \)
2.2.4.7 Photographs - Precaster D

*Figure 2.2.4.7-1. Planetary Pan Mixer (Precaster D)*

*Figure 2.2.4.7-2. Adding Fibers to the Mixer (Precaster D)*
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Precaster Mix Design Characterization

Figure 2.2.4.7-3. Joist Forms (Precaster D)

Figure 2.2.4.7-4 Filling forms with bucket (Precaster D)

Figure 2.2.4.7-5. Filling Forms with an Auger-Fed Delivery Truck (Precaster D)

Figure 2.2.4.7-6. Thin UHPC Panel (Precaster D)
2.2.5 Precaster E

<table>
<thead>
<tr>
<th>Date of Production Trial</th>
<th>August 13-14, 2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weather Conditions</td>
<td>80 °F, sunny, light breeze</td>
</tr>
</tbody>
</table>

2.2.5.1 Batching

<table>
<thead>
<tr>
<th>Mixer Type</th>
<th>Horizontal twin-shaft (Figure 2.2.5.1-3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixer Capacity</td>
<td>6 cubic yards</td>
</tr>
<tr>
<td>Batch Volume</td>
<td>2.5 cubic yards</td>
</tr>
<tr>
<td>Number of Batches Produced</td>
<td>3: 2 batches with UNL-developed mix design (UNL-1 and UNL-2) 1 batch with project-developed mix design (PCI-1)</td>
</tr>
</tbody>
</table>

Batching Notes

- Material was batched in a "buttered" mixer.
- Aggregate moisture content was measured by burn-off. Batch water was adjusted based on measured moisture content; however, a portion of the design HRWR was held back from each batch to be added later, if needed.
- Silica fume was added by bag.
- Fibers were added through sieve (Figure 2.2.5.1-2).
- HRWR was substituted by a workability-retaining admixture for each batch.
- Ice was used at a nominal 30% substitution of water, by weight, for all three batches. Ice was added from 20-lb. bags.
- Samples were made from batches UNL-2 and PCI-1.

2.2.5.2 Mixture Design (As Batched)

<table>
<thead>
<tr>
<th>Material</th>
<th>UNL-2 (per cubic yard)</th>
<th>PCI-1 (per cubic yard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume Batched</td>
<td>2.5 yd³</td>
<td>2.5 yd³</td>
</tr>
<tr>
<td>Type I/II Cement</td>
<td>1,207 lb.</td>
<td>1,424 lb.</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>160 lb.</td>
<td>350 lb.</td>
</tr>
<tr>
<td>Slag</td>
<td>585 lb.</td>
<td>--</td>
</tr>
<tr>
<td>Sand</td>
<td>1,637 lb.</td>
<td>1,904 lb.</td>
</tr>
<tr>
<td>Water</td>
<td>206 lb.</td>
<td>190 lb.</td>
</tr>
<tr>
<td>Ice</td>
<td>100 lb.</td>
<td>80 lb.</td>
</tr>
<tr>
<td>High-Range Water Reducer (HRWR) - 29% solids</td>
<td>673 fl. oz.</td>
<td>1,158 fl. oz.</td>
</tr>
<tr>
<td>Workability Retaining Admixture - 30% solids</td>
<td>98 fl. oz.</td>
<td>110 fl. oz.</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>265 lb.</td>
<td>265 lb.</td>
</tr>
<tr>
<td>w/b</td>
<td>0.176</td>
<td>0.187</td>
</tr>
<tr>
<td>w/c</td>
<td>0.284</td>
<td>0.233</td>
</tr>
<tr>
<td>s/c</td>
<td>1.36</td>
<td>1.34</td>
</tr>
<tr>
<td>% silica fume by wt. cement</td>
<td>13%</td>
<td>25%</td>
</tr>
<tr>
<td>% slag by wt. cement</td>
<td>48%</td>
<td>--</td>
</tr>
<tr>
<td>HRWR dosage, fl. oz./cwt binder</td>
<td>34.5</td>
<td>65.3</td>
</tr>
<tr>
<td>Workability retaining admixture dosage, fl. oz./cwt binder</td>
<td>5.0</td>
<td>6.2</td>
</tr>
</tbody>
</table>

2.2.5.3 Batching Sequence

<table>
<thead>
<tr>
<th>Step</th>
<th>Approximate Duration, min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Add sand and silica fume.</td>
</tr>
<tr>
<td>2.</td>
<td>Mix for 3 min. (UNL-2) or 4 min. (PCI-1)</td>
</tr>
<tr>
<td>3.</td>
<td>Add cement and slag, as applicable.</td>
</tr>
<tr>
<td>4.</td>
<td>Mix for 2 min. (UNL-2) or 6 min. (PCI-1)</td>
</tr>
<tr>
<td>5.</td>
<td>Add water and ice.</td>
</tr>
</tbody>
</table>
Implementation of UHPC in Long-Span Precast Pretensioned Elements

<table>
<thead>
<tr>
<th>Step</th>
<th>Precaster Mix Design Characterization</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.</td>
<td>Mix 4 min.</td>
</tr>
<tr>
<td>7.</td>
<td>Add HRWR and workability retaining admixture.</td>
</tr>
<tr>
<td>8.</td>
<td>Mix to turn, then sample and measure flow spread.</td>
</tr>
<tr>
<td>9.</td>
<td>Add fibers.</td>
</tr>
<tr>
<td>10.</td>
<td>Mix 5 min. (UNL-2) or 3 min. (PCI-1), then sample, measure flow spread.</td>
</tr>
<tr>
<td></td>
<td>Total mixing time</td>
</tr>
</tbody>
</table>

**2.2.5.4 Fresh UHPC Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Target</th>
<th>UNL-2</th>
<th>PCI-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature at discharge</td>
<td>65 to 85 °F</td>
<td>84 °F</td>
<td>83 °F</td>
</tr>
<tr>
<td>Temperature at point of placement</td>
<td>65 to 85 °F</td>
<td>85 °F</td>
<td>85 °F</td>
</tr>
<tr>
<td>Flow spread before fiber addition</td>
<td>8 to 10 in.</td>
<td>&gt; 12 in.</td>
<td>11 in.</td>
</tr>
<tr>
<td>Flow spread at discharge</td>
<td>8 to 10 in.</td>
<td>8 1/2 in.</td>
<td>8 3/4 in.</td>
</tr>
<tr>
<td>Flow spread at point of placement</td>
<td>8 to 10 in.</td>
<td>&lt; 5 in.</td>
<td>5 in.</td>
</tr>
</tbody>
</table>

**Fresh UHPC Observations**
- Fibers were added with sand for mix UNL-1. Fibers balled in mixer after being added over a screen and mix was discarded.
- Fiber segregation and settlement was observed for mix UNL-2, which had a flow spread greater than 12 in. Flow spread dropped from 8 1/2 in. at discharge to 5 in. after approximately 20 minutes. Fiber balls were also observed. Mix was not self-consolidating during placement.
- Rapid drop-off in flow was observed for mix PCI-1. Drop-off may be due to low water content. Mix was not self-consolidating at end of placement. No fiber balls or clumps were observed.

**2.2.5.5 Element Production**

<table>
<thead>
<tr>
<th>Element(s) Produced</th>
<th>Both mixes: one waffle slab each with and without bars (<a href="#">Figure 2.2.5.1-3</a>); one slab each with truss bar (<a href="#">Figure 2.2.5.1-4</a>) PCI-1: two additional 1-inch slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method for Transporting Material to Forms</td>
<td>Front-loader mixing truck</td>
</tr>
<tr>
<td>Consolidation</td>
<td>None</td>
</tr>
<tr>
<td>Finishing</td>
<td>Surface tamped with 2x4, then finished with a spiked roller.</td>
</tr>
<tr>
<td>Curing</td>
<td>Under plastic</td>
</tr>
<tr>
<td>Thermal Treatment</td>
<td>None</td>
</tr>
</tbody>
</table>

**Production Notes**
- Placement followed leading edge ([Figure 2.2.5.1-5](#)).
- Plastic sheeting was applied to surfaces within 5 min. of finishing to reduce elephant skin formation ([Figure 2.2.5.1-6](#)).
### 2.2.5.6 Hardened UHPC Properties

| Test Specimens Produced | 12 3x6 cylinders cured in lab (6 each mix)  
|  | 12 3x6 cylinders cured with product (6 each mix)  
|  | 6 4x4x14 beams cured in lab (3 each mix)  
|  | 6 4x4x14 beams cured with product (3 each mix)  
|  | 4 unstressed strand bond samples with 4x8 inch cross-section, 6, 12, 18, and 24 inches in length, and containing three 0.7-inch strands spaced at 2 inches at mid-height; and 4 stressed strand bond samples with same cross sections, containing three 0.6-inch strands spaced at 2 inches at mid-height (PCI-1 mix only) |
| 28-day compressive strength (ASTM C1856, performed on 3x6 cylinders) | Mix UNL-2 (average of 3): 17,310 psi  
|  | Mix PCI-1 (average of 3): 18,020 psi |
| 28-day flexural strength (ASTM C1856) | Mix UNL-2 (average of 3):  
|  | First-peak flexural strength, $f_{fc}$: 2,140 psi  
|  | Peak flexural strength, $f_{fu}$: 3,620 psi (170% of $f_{fc}$)  
|  | Residual flexural strength at L/300: 168% of $f_{fc}$  
|  | Residual flexural strength at L/150: 136% of $f_{fc}$ |
|  | Mix PCI-1 (average of 3):  
|  | First-peak flexural strength, $f_{fc}$: 2,110 psi  
|  | Peak flexural strength, $f_{fu}$: 3,120 psi (148% of $f_{fc}$)  
|  | Residual flexural strength at L/300: 143% of $f_{fc}$  
|  | Residual flexural strength at L/150: 110% of $f_{fc}$ |

### 2.2.5.7 Photographs - Precaster E

*Figure 2.2.5.1-1. Horizontal Twin-Shaft Mixer (Precaster E)*  
*Figure 2.2.5.1-2. Adding Fibers to the Mixer (Precaster E)*
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Precaster Mix Design Characterization

Figure 2.2.5.1-3. Waffle Slab Forms (Precaster E)

Figure 2.2.5.1-4. Truss Slab Forms (Precaster E)

Figure 2.2.5.1-5. Filling Forms from a Mixing Truck (Precaster E)

Figure 2.2.5.1-6. Covering the Surface with a Plastic Sheet (Precaster E)
2.2.6 Precaster F

Date of Production Trial | April 11-12, 2019
Weather Conditions | Indoors, 75 °F (24 °C) on 4/11, 68 °F (20 °C) on 4/12

2.2.6.1 Batching

| Mixer Type | Rotoconix® Triple-Effect Concrete Mixer (Figure 2.2.6.7-1) |
| Mixer Capacity | 0.75 m³ (1 yd³) |
| Batch Volume | 0.4 m³ (0.5 yd³) |
| Number of Batches Produced | 5: 2 batches on 4/11 and 3 batches on 4/12 |

Batching Notes:
- Powder materials (cement, silica fume, slag and sand) were pre-weighed into supersacks prior to batching.
- All aggregates contained in the pre-mix were oven dry.
- Materials were batched in a clean mixer.
- Water and chemical admixtures were manually weighed and added from buckets.
- Fibers were manually weighed and added through a dispensing chute.
- 75% of mixing water was replaced with ice for batches on 4/12.

2.2.6.2 Mixture Design

Precaster F had an independently-established UHPC mixture, separate from the efforts of this research project. As such, laboratory samples for Precaster F were produced separately from the in-plant production trials. Accordingly, the nominal mixture proportions are presented below.

<table>
<thead>
<tr>
<th>Material</th>
<th>Nominal Proportions (per cubic yard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type GUL Cement</td>
<td>1,075 lb.</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>340 lb.</td>
</tr>
<tr>
<td>Slag</td>
<td>357 lb.</td>
</tr>
<tr>
<td>Sand</td>
<td>1,770 lb.</td>
</tr>
<tr>
<td>Water</td>
<td>228 lb.</td>
</tr>
<tr>
<td>High-Range Water Reducer (HRWR) - 40% solids</td>
<td>966 fl. oz.</td>
</tr>
<tr>
<td>Workability-Retaining Admixture - 21% solids</td>
<td>192 fl. oz.</td>
</tr>
<tr>
<td>Steel Fiber</td>
<td>266 lb.</td>
</tr>
<tr>
<td>w/b</td>
<td>0.146</td>
</tr>
<tr>
<td>w/c</td>
<td>0.240</td>
</tr>
<tr>
<td>s/c</td>
<td>1.65</td>
</tr>
<tr>
<td>% silica fume by wt. cement</td>
<td>32%</td>
</tr>
<tr>
<td>% slag by wt. cement</td>
<td>33%</td>
</tr>
<tr>
<td>HRWR dosage, fl. oz./cwt binder</td>
<td>54.5</td>
</tr>
<tr>
<td>Workability-retaining admixture dosage, fl. oz./cwt binder</td>
<td>10.8</td>
</tr>
</tbody>
</table>

2.2.6.3 Batching Sequence

<table>
<thead>
<tr>
<th>Step</th>
<th>Approximate Duration, min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Add pre-weighed dry materials to mixer.</td>
<td>1</td>
</tr>
<tr>
<td>2. Dry mix for 1 minute.</td>
<td>1</td>
</tr>
<tr>
<td>3. Add mixing water, ice, and chemical admixtures.</td>
<td>1</td>
</tr>
<tr>
<td>4. Mix until turn.</td>
<td>5.5</td>
</tr>
<tr>
<td>5. Add fibers.</td>
<td>3</td>
</tr>
<tr>
<td>6. Continue mixing for 3 minutes until homogenous.</td>
<td>3</td>
</tr>
<tr>
<td>7. Sample and measure flow spread, then discharge into bucket.</td>
<td>3.5</td>
</tr>
<tr>
<td>Total mixing time</td>
<td>18 min.</td>
</tr>
</tbody>
</table>
2.2.6.4 Fresh UHPC Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Target</th>
<th>Batch #412-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature at discharge</td>
<td>65 to 85 °F</td>
<td>63 °F (17 °C)</td>
</tr>
<tr>
<td>Temperature at point of placement</td>
<td>65 to 85 °F</td>
<td>62 °F (16.8 °C)</td>
</tr>
<tr>
<td>Flow spread before fiber addition</td>
<td>8 to 10 in.</td>
<td>Not measured</td>
</tr>
<tr>
<td>Flow spread at discharge</td>
<td>8 to 10 in.</td>
<td>9-1/4 in. (235 mm)</td>
</tr>
<tr>
<td>Flow spread at point of placement</td>
<td>8 to 10 in.</td>
<td>Not measured</td>
</tr>
</tbody>
</table>

Fresh UHPC Observations
- 5 percent additional water was added to improve workability.
- Mix retained workability for approximately 45 minutes.

2.2.6.5 Element Production

Element(s) Produced
1 “tabletop” shear panel element (4/11; Figures 2.2.6.7-3 and 2.2.6.7-4) and 2 H-piles (4/12; Figures 2.2.6.7-5 and 2.2.6.7-6)

Method for Transporting Material to Forms
Bucket (Figure 2.2.6.7-2)

Consolidation
None

Finishing
Level finished surfaces with spiked roller (Figure 2.2.6.7-4); no finishing for H-piles

Curing
Cover with plastic, then heat form (no controller is used)

Thermal Treatment
Expose to steam after stripping

Production Notes
- Material was placed behind the leading edge.
- Two batches were deposited into bucket. The first batch was covered with wet burlap while the second batch was being produced.

2.2.6.6 Hardened UHPC Properties

All testing of materials for Precaster F was performed on laboratory-cured specimens.

Test Specimens Produced
- 6 4x8 cylinders cured in lab
- 3 4x4x14 beams cured in lab
- 4 stressed and 4 unstressed strand bond samples with 6x6-inch square cross-section, 6, 12, 18, and 24 inches in length, and containing three 0.6-inch strands spaced at 2 inches at mid-height

28-day compressive strength (ASTM C1856, performed on 3x6 cylinders)
Lab-cured specimens (average of 3): 18,160 psi

28-day flexural strength (ASTM C1856)
Lab-cured specimens (average of 3):
- First-peak flexural strength, $f_{fc}$: 1,760 psi
- Peak flexural strength, $f_{fu}$: 2,440 psi (139% of $f_{fc}$)
- Residual flexural strength at L/300: 121% of $f_{fc}$
- Residual flexural strength at L/150: 98% of $f_{fc}$
2.2.6.7 Photographs - Precaster F

Figure 2.2.6.7-1. Concrete Mixer (Precaster F)

Figure 2.2.6.7-2. A Bucket used to Transport UHPC (Precaster F)

Figure 2.2.6.7-3. Filling the Tabletop Panel using a Bucket (Precaster F)

Figure 2.2.6.7-4. Leveling the Surface with a Spiked Roller (Precaster F)
Figure 2.2.6.7-5. H-Pile Forms (Precaster F)

Figure 2.2.6.7-6. Filling H-Pile Forms using a Bucket (Precaster F)
2.2.7 Discussion of Observations

Key observations and lessons learned are summarized below.

2.2.7.1 Batching and Mixing

- **Materials Handling**: At each plant, materials that are not used for current concrete production were added manually to the mixer, which generally increased batching times for each mix. Fiber addition was typically the slowest step of the mixing process. At some plants, admixture dispensing through conventional equipment also extended batching times due to the limited throughput of the equipment.

- **Batching Sequence**: Batching sequences varied among the different plants. Through the various in-plant trials, it was observed that the addition of powder materials, especially silica fume, to pre-wetted aggregates increased the tendency for “balls” to form in the mixer. In some cases, these balls did not break up during the full mixing cycle, but the number of powder balls reduced with increased mixing time with sand, and with increased shearing brought about by the addition of the steel fibers. It was also observed that mixing times were reduced and flow spread generally increased when a portion (about 30 percent) of the HRWR was added with the sand during the “dry-mixing” stage of the batching sequence, rather than adding the HRWR with the mixing water.

- **Effectiveness of HRWR**: The effectiveness of the high-range water reducing admixtures (HRWRs) in achieving the target performance is a function of the particular mixer type, mixture constituents, and water-binder ratio. As observed during the laboratory trial batching, certain chemical admixtures were more effective with certain mixture designs than others, due primarily to the particular constituent material combinations used in the mixture. Large dosages of HRWR—significantly greater than the manufacturers’ recommended dosages for conventional concrete—were required for each mix. Reductions in HRWR dosage in the in-plant trials relative to the laboratory trial batches were possible for some plants due to the greater efficiency of the mixer.

- **Benefits of Ice**: Trial batches at Precaster F were produced both with and without ice. Production personnel commented that better workability characteristics (e.g., flow spread, flow retention, and the tendency for skin formation) were observed when a portion of the mixing water was replaced with ice.

- **Initial Condition of the Mixer**: Less variability in batching times and initial flow spread was observed in plants where the mixer was in a consistent initial condition (e.g., “buttered” or “clean”) at the start of each batch. Coating the mixer with a small “mortar” batch composed of the raw materials was beneficial.

- **Fiber Dispersion**: Fiber “clumping” and “balling” were consistent challenges observed at most of the plants. Fiber “clumps” were defined as agglomerates of fibers and wet UHPC paste (Figure 2.2.7.1-1), while fiber “balls” were defined as relatively dry agglomerates of fibers and dry materials (Figure 2.2.7.1-2). The fibers had the same properties, but different lengths (½ and ¾-in.). This effected the tendency for clumping.

The fiber clumps in the mixed UHPC appear to have been agglomerations of fibers as they came out of the bags and were introduced into the mixer; even after significant mixing action, these clumps did not break up. Fiber “clumps” were commonly observed when the fibers were rapidly added directly to the mixer from the bags. Less “clumping” was observed when fibers were added slowly to the mixer, and especially when they were added over a sieve or other device intended to facilitate fiber separation. Less “clumping” was also observed when more shearing action was provided to the mix, either through more effective mixing action of the mixer or through the stiffer consistency (i.e., lower flow) of the mix.

Fiber “balls” were observed at only two precasters and occurred when the fibers were introduced during the dry mixing stage. The fibers and dry paste formed balls, the interior of which remained dry through the full mixing cycle. Additional mixing while the mixture was relatively stiff (at around a 5-inch flow spread) appeared to facilitate breaking up of these fiber balls.
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Batch Adjustments: A portion of the mixing water (about 10 percent) was held back during the in-plant trials at Precaster D. This tail water was typically added to the mixer after the fibers had been dispersed to increase the flow spread to the target range. Fibers appeared to disperse more effectively when the mixture was less fluid; however, the tail water was typically needed to provide sufficient flow for the production of the precast members.

2.2.7.2 Material Transport & Placement

Transport Methods: Fresh UHPC was transported to the mixer using buckets (Precaster F and Precaster D), mixer trucks (Precaster C), and auger-fed delivery trucks (Precaster A and Precaster D). Placement via a bucket has limitations in both the bucket capacity (typically only a few cubic yards, at most) and in the static conditions in which the material is held. UHPC will tend to become more thixotropic, and lose workability, under static conditions. Placement from a mixer truck has the advantage of keeping the material in motion during transit, which facilitates longer working time. In addition, the large capacity allows combination of multiple batches. Placement using an auger-fed delivery truck does not provide the same advantages as the mixing truck with respect to workability retention; however, the auger feed does provide additional mixing action to “re-activate” the UHPC prior to placement, which improves workability relative to a static, bucket condition.

Placement: It is generally advantageous if UHPC can be placed into formwork in a continuous placement from a single point of placement; however, due to the complex geometry and shallow clear covers for many of the mock-up elements produced, such placement was difficult - and at times, not practical or possible. In a trial mock-up element produced at Precaster C using the first days’ trial batches, it was discovered that the material did not fully flow around the void former in the center of the element, resulting in a large voided area at the base of the mock-up element. Therefore, for the production on the second day, placement was generally accomplished by depositing new material behind the leading edge. Successful placements at the other precast plants were also typically accomplished by following the leading edge.

Speed of Placement: It was generally observed that placements that were completed quickly (within about 10 minutes) were typically easier to finish and had less tendency to form a surface crust (“elephant skin”) compared to placements that took longer to complete. Faster placement times were possible for mixes with greater flow spreads. It should be noted that faster placements may also entrap more air within the forms, particularly if the forms are not consolidated by vibration after placement.

Large Product Strategies: The trial elements placed varied in volume from 1 to 6 cubic yards of material, and required one or two batches of material to fully fill the forms. When multiple batches are required, it was observed that combining batches in a bucket, with wet burlap covering the first batch until the second batch could be discharged, or combining batches in a ready-mix truck presented viable options, provided that the first batch of material remains workable while subsequent batches are being produced. Alternatively, production of large products may also be facilitated by incorporating joints into the design such that a smaller volume of material can be placed at a time.
2.2.7.3 Workability Retention

- **Workability Retention**: Although only small precast pieces were produced during the plant trials, workability retention was evaluated over the duration of each pour in anticipation of larger-scale production. Workability retention was defined as the duration of time after discharge from the mixer that the flow spread remained above 7 inches. The workability retention of each plant’s mixture is summarized in Table 2.2.6.3-1. All plants, with the exception of Precaster E, were able to achieve at least 40 minutes of workability retention with their current mixture proportions and handling methods. The limited workability retention observed for Precaster E is believed to be related to a reduced water content, and additional work will be performed to ensure that the mixture has sufficient workability retention for placement. Workability retention appeared to be longest at plants where the material was maintained at cooler temperatures (i.e., Precaster A) and/or agitated during transit (i.e., Precaster C).

<table>
<thead>
<tr>
<th>Precaster</th>
<th>Precaster A</th>
<th>Precaster B</th>
<th>Precaster C</th>
<th>Precaster D</th>
<th>Precaster E</th>
<th>Precaster F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow spread at discharge</td>
<td>9-3/8 in.</td>
<td>8-3/4 in.</td>
<td>10-1/4 in.</td>
<td>10-3/4 in.</td>
<td>8-1/2 in.</td>
<td>8-3/4 in.</td>
</tr>
<tr>
<td>Concrete temperature at discharge</td>
<td>63 °F</td>
<td>81 °F</td>
<td>91 °F</td>
<td>83 °F</td>
<td>84 °F</td>
<td>63 °F</td>
</tr>
<tr>
<td>Material handling method</td>
<td>Auger-fed Delivery Truck (static w/ auger)</td>
<td>Mixing Truck and Bucket (agitated/static)</td>
<td>Mixing Truck (agitated)</td>
<td>Auger-fed Delivery Truck (static w/ auger)</td>
<td>Mixing truck (agitated)</td>
<td>Bucket (static)</td>
</tr>
<tr>
<td>Workability retention</td>
<td>&gt; 40 min. (10 in. flow at end of placement)</td>
<td>50 min.</td>
<td>&gt; 90 min.</td>
<td>50 min.</td>
<td>&lt; 20 min.</td>
<td>45 min.</td>
</tr>
</tbody>
</table>

2.2.7.4 Finishing and Curing

- **Elephant Skin**: An "elephant skin" crust formed on the top surfaces of most of the UHPC elements placed. This skin forms as a result of near-surface drying of the UHPC and is exacerbated by high concrete temperatures, high ambient temperatures, windy conditions, and the UHPC mixture design. It was observed at multiple plants that the elephant skin entrapped air bubbles beneath the surface, creating a voided layer just beneath the surface (Figures 2.2.6.4-1 and 2.2.6.4-2). Cooler concrete temperatures, covering the concrete with plastic sheeting shortly after finishing, and the use of certain chemical admixtures such as RMAs helped reduce the tendency for the elephant skin to form on the surface of the elements.
Precaster Mix Design Characterization

- **Spiked Rollers**: Spiked rollers were used at Precaster A, Precaster E, and Precaster F to break up surface skins and enable some finishing of the surfaces. The rollers made it possible to level the surfaces and free trapped air voids without causing surface tears. Some small depressions remained on the surface after rolling.

![Figure 2.2.7.4-3. Depressions in the Concrete Surface after Finishing with a Spiked Roller](image)

- **Evaporation Reducer**: An evaporation reducer was applied to the surface of the UHPC elements produced by Precaster A to facilitate finishing and to reduce surface drying.

- **Heat of Hydration**: Most of the participating precasters cured their mock-up elements under tarps or plastic sheeting, without the use of supplemental heat. However, the high heat of hydration of the UHPC mixtures resulted in relatively high internal concrete temperatures, as outlined in Table 2.2.6.4-1. The potential for thermal cracking may need to be considered.

<table>
<thead>
<tr>
<th>Precaster</th>
<th>Precaster A</th>
<th>Precaster B</th>
<th>Precaster C</th>
<th>Precaster D</th>
<th>Precaster E</th>
<th>Precaster F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ambient Temp., °F</td>
<td>50 - 60 °F</td>
<td>55 - 75 °F</td>
<td>87 °F</td>
<td>65 - 85 °F</td>
<td>80 °F</td>
<td>68 °F</td>
</tr>
<tr>
<td>Concrete Temp. at Discharge, °F</td>
<td>63 °F</td>
<td>81 °F</td>
<td>91 °F</td>
<td>83 °F</td>
<td>83 – 84 °F</td>
<td>63 °F</td>
</tr>
<tr>
<td>Max. Product Temperature, °F</td>
<td>156 °F</td>
<td>179 °F</td>
<td>180 °F</td>
<td>--</td>
<td>106 °F</td>
<td>Not reported</td>
</tr>
<tr>
<td>Max. Test Cylinder Temp. (lab cured), °F</td>
<td>Not reported</td>
<td>111 °F</td>
<td>104 °F</td>
<td>94 °F</td>
<td>106 °F</td>
<td>Not reported</td>
</tr>
<tr>
<td>Max. Test Cylinder Temp. (match cured)*, °F</td>
<td>156 °F</td>
<td>148 °F</td>
<td>157 °F</td>
<td>107 °F</td>
<td>N/A</td>
<td>Not reported</td>
</tr>
</tbody>
</table>

*Match-curing systems used by the individual precasters tended to limit the curing temperature to less than 158 °F, even if higher temperatures were measured in the product.

### 2.3 UHPC Mixture Characterization

#### 2.3.1 Compressive Strength and Flexural Strength Testing

The UHPC mixes produced by each precaster were characterized based on compressive and flexural performance, both in accordance with ASTM C1856. Compressive strength was evaluated at 28 days for samples cured with the product (or match-cured, if possible) and for samples receiving standard laboratory curing at 73.5 ± 3.5 °F and at least 95 percent relative humidity. Flexural performance was also evaluated at 28 days, but only for samples receiving standard laboratory curing. Three samples were tested for each test and curing condition.

The test results for the six precasters are summarized in Table 2.3.1-1 below; results for both mixtures produced by Precaster E are presented. The average flexural performance of the three flexural tests conducted for each precaster are also shown in Figure 2.3.1-1.
Precaster Mix Design Characterization

As shown in the table, mixes from Precaster B and Precaster D, and the UNL mixture from Precaster E, did not meet the performance targets for compressive strength by 28 days based on ambient curing; however, with additional curing time, it is anticipated that the material from Precaster B and E will exceed 18,000 psi in service. With additional curing time, the material from Precaster D may also exceed 18,000 psi, and with additional heat provided post-curing this mix almost certainly would meet this target strength, as shown in the table. The mixture from Precaster A had the highest compressive strength and first peak (first-crack) flexural strength of all mixes produced; however, it was the only mixture to not exhibit strain hardening behavior post-cracking (Figure 2.3.1-1), and had an average peak flexural strength only 105% of the first-peak strength. Based on observations during testing, it is believed that the high fiber-matrix bond strength between the Precaster A UHPC and the 3/4-inch fibers resulted in an increased tendency for failure by fiber breakage rather than fiber elongation and slippage. Shorter fibers or a weaker matrix may promote more strain-hardening behavior for this mixture.

Table 2.3.1-1. Precaster Mixture Performance (Average of 3)

Highlighted values do not meet performance targets at 28 days.

<table>
<thead>
<tr>
<th>Property</th>
<th>Target</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E (UNL)</th>
<th>E (PCI)</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength, 28 days</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lab-cured, psi</td>
<td>&gt; 18,000*</td>
<td>24,190</td>
<td>17,900</td>
<td>18,970</td>
<td>16,290</td>
<td>17,310</td>
<td>18,020</td>
<td>18,160</td>
</tr>
<tr>
<td>Cured with product, psi</td>
<td>&gt; 18,000*</td>
<td>22,840</td>
<td>17,830</td>
<td>19,780</td>
<td>16,660</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Cured 72 hr at 194°F, psi</td>
<td>&gt; 18,000*</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>19,330</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Flexural Strength, 28 days</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>First peak, psi</td>
<td>&gt; 1,500*</td>
<td>2,360</td>
<td>2,190</td>
<td>1,960</td>
<td>2,100</td>
<td>2,140</td>
<td>2,110</td>
<td>1,760</td>
</tr>
<tr>
<td>Peak, psi</td>
<td>&gt; 2,000*</td>
<td>2,470</td>
<td>3,620</td>
<td>3,170</td>
<td>3,250</td>
<td>3,620</td>
<td>3,120</td>
<td>2,440</td>
</tr>
<tr>
<td>Peak, % of first peak</td>
<td>&gt; 125%*</td>
<td>105%</td>
<td>166%</td>
<td>162%</td>
<td>154%</td>
<td>170%</td>
<td>148%</td>
<td>139%</td>
</tr>
<tr>
<td>Residual at L/300, % of first peak</td>
<td>&gt; 90%*</td>
<td>91%</td>
<td>161%</td>
<td>160%</td>
<td>150%</td>
<td>168%</td>
<td>143%</td>
<td>121%</td>
</tr>
<tr>
<td>Residual at L/150, % of first peak</td>
<td>&gt; 75%*</td>
<td>78%</td>
<td>129%</td>
<td>137%</td>
<td>111%</td>
<td>136%</td>
<td>110%</td>
<td>98%</td>
</tr>
</tbody>
</table>

* Targets specified at service, but measured at 28 days.
2.3.2 Shrinkage Testing

Shrinkage was evaluated for each mixture beginning 24 hours after casting through 90 days of age. A separate 1-cubic foot batch of each mixture was produced in the laboratory using the same mixture proportions and following the same general mixing procedures used during the in-plant trials. The flow spread of each mixture was verified to be within 1/2-inch of the flow spread measured during the plant production trials. Compressive strengths were verified to be generally within 200 psi of the compressive strengths measured for the plant-batched mixtures, with the exception of the two mixtures from Precaster E, which were both about 1,500 psi lower in compressive strength than the field-batched mixtures; the reason for this exception is unclear.

Eight 3-inch by 3-inch by 11.25-inch specimens were produced from each batch. All specimens were cured in a standard moist-curing environment at 73.5 ± 3.5 °F and at least 95 percent relative humidity.

After 24 hours of curing or after final set, whichever occurred last, the specimens were stripped from their molds, and an initial length reading was taken. Four of the specimens were sealed with aluminum tape, and the other four specimens were unsealed. The purpose of the sealed specimens is to evaluate the autogenous shrinkage that occurs within the specimens over a 3-month period, while the purpose of the unsealed specimens is to evaluate the total drying and autogenous shrinkage occurring within the specimens under standard laboratory drying conditions.

All eight specimens were stored in a controlled environment at 73.5 ± 3.5 °F and 50 ± 4 percent relative humidity in accordance with ASTM C157. Readings were taken daily for 7 days, weekly for 1 month, and monthly for 3 months.

Preliminary results are presented in Figures 2.3.2-1 through 2.3.2-3. Testing for mixtures from Precaster E is still in progress and complete results will be included in the final Project Report.

Typical magnitudes of total shrinkage reported for UHPC mixtures range from about 500 to 900 microstrain (10^-6 in/in), depending on the mixture proportions and the curing applied (for more discussion, see Section 5.3.1). The mixtures produced in this research effort are generally consistent with these ranges, with Precaster F's mixture showing the least total shrinkage, approximately 400 microstrain after 90 days, and Precaster D's mixture showing the greatest total shrinkage, approximately 800 microstrain after 90 days. As shown in Figures 2.3.2-2 and 2.3.2-3.
Most of this shrinkage is autogenous shrinkage, caused by the hydration of the cementitious materials and the low w/b of the mixtures. Drying shrinkage accounts for approximately 100 to 150 microstrain of the total and is essentially constant after 14 days of drying for all seven mixtures. Some autogenous shrinkage may have occurred after initial set but prior to the start of testing at 24 hours; this shrinkage is not accounted for in the results presented herein but may be 100 microstrain or more (i.e., the total shrinkage measured according to the methods used may be less than the true total shrinkage exhibited by the UHPC mixtures). Additional evaluation of early-age shrinkage using embedded vibrating wire strain gages is planned as part of a parallel research effort being conducted by researchers at The Ohio State University and is intended to provide additional information regarding volumetric changes in UHPC occurring at these very early ages.

Figure 2.3.2-1. Average Measured Total (Autogenous + Drying) Shrinkage for Unsealed Specimens
Precaster Mix Design Characterization

**Figure 2.3.2-2.** Average Measured Autogenous Shrinkage for Sealed Specimens

**Figure 2.3.2-3.** Average Computed Drying Shrinkage, Based on Data Presented in Figure 2.3.2-1 and 2.3.2-2
2.4 Mixing and Placing Productivity in Plant Setting

Most of the plants partnering in this research have powerful mixers and automated batch plants. While one plant has a specialized mixer specifically suited for rapid mixing of UHPC, most of the planetary and twin-shaft type mixers used in the other plants can have adequate energy to turn the concrete fluid in a matter of few minutes from the time the chemicals are added to the mixture.

For the trial batches reported above, the total mixing time was 20-40 min.; however, this does not reflect typical mixing times that may be expected for full-scale production. In addition to the reduced production rate associated with sampling the UHPC for flow testing, the extended mixing times may be attributed to several factors:

1. The batch plants were not set up to automate dispensing of the raw UHPC materials that were not part of typical production (e.g. supplementary materials and in some cases silica fume), and therefore these individual raw materials were added to the mixer by hand.
2. Some batch plants were not equipped to rapidly dispense the large volumes of admixture required for UHPC, and in one instance dosing of the admixture alone took more than 5 min.
3. Some batch plant computer systems were not set up to consider the water content of the admixtures, so external calculations of target batch water were required to compensate for both admixtures and sand moisture condition.
4. Most importantly, none of the precasters had an automatic fiber dispenser and had to screen the fibers by hand, which took 10 to 15 min. per 3 cubic yard batch.

Table 2.4-1 shows typical mixing and precasting cycle for 2.5 cubic yards based on recent trials at one of the plants. The table indicates target sequence durations for full-scale production, after experience is gained from several trials. Note that a long-span precast member may require up to 40 cubic yards and perhaps even more for the 250 ft members aimed to be promoted in this research. If one assumed that the second cycle of mixing starts at the discharge from the mixer (22 min.), the time elapsed after the second cycle will be $22 + 42 = 64\text{ min.}$ and after the third cycle $86\text{ min.}$ and so on. Thus, 40 cubic yards, requiring 16 batches, would take $42 + 15 \times 22 = 372 \text{ min.}$, or 6.2 hours. While this amount of time is possibly within a work shift, it is much longer than the 2 hours or less it takes to produce the same member with conventional concrete.
A possible improvement in efficiency is to pre-blend the cement, silica fume and supplementary cementitious material and feed them in bulk to the mixer. A second possible improvement to supply the fibers with automatic dispenser. If these two improvements, and perhaps others, could reduce the mixing cycle in half to 11 min., then the total mixing and delivery of 40 cubic yards becomes $33 + 15 \times 11 = 198$ min., or 3.3 hours.

Another means for improving production rates is to increase the batch sizes. Most plants participating in these trials batched at about half of their mixer capacity or less. While prudent during the mix development phase to protect the plant motor, observation of mix “amp meters” during the trials suggested that the mixers were operating at well below their power limits throughout the UHPC batch cycle.

The authors of this report believe that it is a matter of time, and capital investment, before the process is streamlined and production is reduced to durations close to current practice.
3 NON-LINEAR STRESS ANALYSIS

3.1 Notation

\( b \) = width of the prism in four-point bending beam test (in.)

\( c \) = distance from the extreme compression fiber to the neutral axis (in.)

\( E_c \) = modulus of elasticity (ksi)

\( f_c \) = flexural compressive stress (ksi)

\( f_t \) = flexural tensile stress (ksi)

\( f_{tu} \) = peak flexural tensile strength (ksi)

\( f_t \) = uniaxial tensile strength (ksi)

\( h \) = depth of the prism in four-point bending beam test (in.)

\( L \) = span length in third-point loading beam test (in.)

\( M \) = maximum bending moment in four-point bending beam test (kip-in.)

\( \varepsilon_b \) = tensile strain at the bottom of the prism in four-point bending beam test (in./in.)

\( \varepsilon_c \) = compressive strain at the top of the prism in four-point bending beam test (in./in.)

\( \phi \) = curvature of the prism in four-point bending beam test (1/in.)

3.2 Introduction

The high structural performance of UHPC is attributed to its superior tensile strength. While the flexural tensile strength of UHPC is needed to check the cracking of the structural members subjected to flexure forces, the uniaxial tensile strength is needed to determine the strength of these members in direct tension. However, there is no standard test to quantify the uniaxial tensile strength. Some researchers (Reineck and Frettlöh (2010) and Graybeal and Baby (2013)) have proposed suitable uniaxial tensile tests. However, the bending test of a beam with third-point loading (ASTM C1609) provides a simpler and just as reliable alternative to these uniaxial tensile tests (López et al., 2015). Many international codes and specifications (NF P 18-710 (2016), SIA 2052 (2016), DAfStb (2017), and CSA S6 Annex 8.1 (not published yet)) also specify the flexural beam test as the standard test to determine the tensile properties of UHPC. Some practices specify conversion factors while others apply inverse analysis to the beam test results. López et al. (2016) has developed a simplified model utilizing the inverse analysis. This model is recommended by CSA S6 Annex 8.1.

3.3 Inverse-Analysis

The inverse analysis is a process of determining the tensile properties of UHPC utilizing the third-point loading test. The development of a moment-curvature diagram that utilizes the idealized stress-strain relationship is proposed in this research, see Figure 3.3-1. The idealized relationship is represented by a bilinear diagram. The first part of this diagram has a slope equal to the modulus of elasticity, determined in accordance with ASTM C1856. At the limit of elasticity, the value is set equal to the tensile strength, \( f_t \), and remains constant as the strain increases to the ultimate tensile strain of 0.004. To obtain a single point on the moment-curvature diagram, a value of the strain at the extreme compression fiber, \( \varepsilon_c \), is selected. Figure 3.3-2 shows different stress distributions over the depth of the beam, which are dependent on the strain values at various distances from the neutral axis. Through an iterative process, the neutral axis location is changed until the tension and compression forces are balanced. Then the curvature of the beam is determined by dividing the compression strain, \( \varepsilon_c \), by the depth of the compression zone. The moment is determined by summing up the products of the force components (tension and compression) times the distance to a fixed point in the section, say the top fiber. These steps are repeated for different values of compression strain to obtain multiple points. The corresponding moments and curvatures are plotted until the ultimate moment is captured from the generated moment-curvature diagram. This procedure should be repeated with different values of the tensile strength, \( f_t \). The actual tensile strength, \( f_t \), should be selected such that the ultimate moment is equal to that obtained from the actual bending test.
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Non-Linear Stress Analysis

Linear Analysis

\[ f_{fu} = MOR = \frac{P_u L}{bh^2} \]

Assume Value of \( f_t \)
- Perform Non-linear Analysis

Repeat Non-linear Analysis with different Values of \( f_t \)

Idealized Tensile Stress-Strain Diagram

Figure 3.3-1. Flow Chart to Develop Idealized Tensile Stress-Strain Diagram
The accuracy of this procedure was evaluated utilizing the same data reported by Graybeal and Baby (2019), in which they assessed different tension tests to determine the tensile properties of UHPC. They also evaluated different inverse analysis approaches (Baby et al. (2012a and 2012b), Rigaud et al. (2011), and Qian and Li (2008)) for the four-point bending test results. It was reported that a larger prism cross section provides a higher level of accuracy in the tensile response. Therefore, only Specimen B2-B, with a 4 in. by 4 in. cross section, with modulus of elasticity of 8160 ksi (Graybeal and Baby, 2019), was used for comparison.

The procedure proposed in this research was conducted to determine the tensile strength, \( f_t \), corresponding to an equivalent flexural tensile strength, \( f_{tu} = \frac{6M}{bh^2} \) of 3.82 ksi, as reported by Graybeal and Baby (2019). By utilizing the proposed approach, the tensile strength was determined to be 1.55 ksi, while the average tensile strength for the various approaches reported by Graybeal and Baby (2019), is 1.51 ksi. Figure 3.3-3 shows a comparison between the proposed approach and the various approaches referenced.

**Figure 3.3-2. Strain and Stress Distribution on Rectangular Section under Bending Test (ASTM C1609)**
With a good correlation obtained in the tensile stress-strain relationship, the inverse analysis was then performed on the same 4 in. by 4 in. prism. The analysis utilized different values of tensile strength to generate the moment-curvature diagram. The iterative process of selecting tensile strengths that would result in an equivalent flexural tensile strength of 2.0 ksi, as specified in this research as a minimum requirement, resulted in a tensile strength of 0.80 ksi. The modulus of elasticity was assumed as the specified minimum value of 6,500 ksi. Figure 3.3-4 shows the moment-curvature relationship using the tensile strength of 0.80 ksi. The ultimate bending moment is equal to 1.82 kip-ft for an equivalent tensile strength of 2.0 ksi.
3.4 Proposed Tentative Design Recommendations

The minimum criteria proposed in the preceding sections should be met. The four-point bending test is considered a reliable test when determining the tensile strength of UHPC. Figure 3.4-1 shows the proposed idealized tensile stress-strain diagram for UHPC. Conservatively, a tensile strength of 0.75 ksi was selected instead of 0.8 ksi. The tensile strength and ultimate strain are limited to 0.75 ksi and 0.004, respectively, for structural design.

![Tensile Stress-Strain Diagram](image)

**Figure 3.4-1. Proposed Idealized Tensile Stress-Strain Diagram**

3.5 References


[www.astm.org](http://www.astm.org)


[www.astm.org](http://www.astm.org)


Implementation of UHPC in Long-Span Precast Pretensioned Elements

Non-Linear Stress Analysis


4 FLEXURAL DESIGN

4.1 Notation

- $A_c$ = total area of the composite section (in.$^2$)
- $A_g$ = area of cross section of the noncomposite precast beam (in.$^2$)
- $A_{ps}$ = area of prestressing steel on the flexural tension side of the member (in.$^2$)
- $A_t$ = area of transformed section at transfer (in.$^2$)
- $b$ = the average width of the specimen at the fracture, as oriented for testing in accordance with ASTM C1609 (in.); design width (in.)
- $c$ = distance from the extreme compression fiber to the neutral axis (in.)
- $d$ = the average depth of specimen at the fracture, as oriented for testing in accordance with ASTM C1609 (in.)
- $d_p$ = distance from extreme compression fiber to the centroid of the prestressing tendons (in.)
- $d_t$ = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement measured along the centerline of the web (in.)
- $E_c$ = modulus of elasticity of UHPC (ksi)
- $e_{pc}$ = eccentricity of strands with respect to the centroid of the composite section (in.)
- $e_{pg}$ = the distance between the center of gravity of bottom strands and the bottom concrete fiber of the beam, gross section (in.)
- $e_{ti}$ = eccentricity of strands with respect to the transformed section at transfer (in.)
- $f_b$ = bottom fiber normal stresses due to prestressing and external loads under the service limit state (ksi)
- $f_f$ = the tensile stress corresponding to certain load during the flexural strength test in accordance with ASTM C1609 (ksi)
- $f_{fc}$ = first-peak (first crack) flexural strength (ksi)
- $f_{fu}$ = peak (ultimate) flexural strength (ksi)
- $f_c$ = stress of concrete in extreme compressed fiber (ksi)
- $f_{c'}$ = compressive strength of concrete for use in design (ksi)
- $f_{s1}$ = required concrete compressive strength at transfer (ksi)
- $f_p$ = peak tensile strength (ksi)
- $f_{ps}$ = average stress in prestressing steel at the time for which the nominal resistance of member is required (ksi)
- $f_t$ = the tensile stress corresponding to certain loads during uniaxial tensile tests (ksi); top fiber normal stresses due to prestress and external loads under the service limit state (ksi)
- $f_{tu}$ = ultimate tensile strength (ksi)
- $f_{tc}$ = first-crack tensile strength (ksi)
- $h$ = overall thickness or depth of a member (in.)
- $I_c$ = moment of inertia of the composite section (in.$^4$)
- $I_g$ = moment of inertia about the centroid of the noncomposite precast beam (in.$^4$)
- $L$ = the span length of the prism during the flexural test in accordance with ASTM C1609 (in.)
- $L_f$ = Fiber length (in.)
- $M$ = midspan moment due to external applied load (kip-in.)
- $M_g$ = unfactored bending moment due to the beam self-weight (kip-in.)
- $M_{LL}$ = unfactored bending moment due to live load (kip-in.)
- $M_{ps}$ = flexural strength with fibers ignored (kip-in.)
- $M_{s2}$ = flexural strength with fibers considered (kip-in.)
- $M_s$ = unfactored bending moment due to slab and haunch weights (kip-in.)
- $M_{SID}$ = unfactored bending moment due to super-imposed dead loads (kip-in.)
- $M_u$ = factored moment at the section (kip-in.)
- $P$ = the load corresponding to certain deflection during the flexural strength test in accordance with ASTM C1609 (kips); the load corresponding to certain deflection during the compressive strength test in accordance with ASTM C39 (kips)
- $P_{ps}$ = total prestressing force before transfer (kips)
- $P_{pp}$ = total prestressing force after early-age shrinkage losses (kips)
4.2 Introduction

The flexural behavior may be considered as one of the clearest characteristics of UHPC. Ultimate limit states should be checked for all flexural members to satisfy the required capacity; however, the service limit states may control the design of prestressed members to assure no cracks under service loads. Unlike conventional concrete, the stresses in the UHPC section under service load cannot directly be compared with the standard modulus of rupture obtained from third-point loading prism tests. The strain hardening behavior of UHPC produces ultimate moments at failure larger than 1.2 the cracking moment at first crack (Gowripalan and Gilbert, 2000). This, in turn, introduces the importance of capturing the first crack stress during the standard test of flexural performance.

More researchers and international guidelines and practices are focused on the bending moment modeling of UHPC. All are in agreement that the strain compatibility and force equilibrium are the best approach to determine the moment capacity of the UHPC section with or without reinforcing. This process assumes that the section is still plane after bending and the stress-strain relationship is linear over the entire depth of the section. The difference between one practice to another is specifying the stress distribution diagram in compression and tension zones. Unlike conventional concrete, the concrete tensile contribution is considered in strength determination, especially, for sections without reinforcement. Therefore, some recommendations drop the requirement of minimum flexural reinforcement accounting solely on the UHPC. On the other hand, most of the guidelines propose the use of linear or bilinear compression stress distribution instead of Whitney’s stress block. This might be attributed to the common practice in Europe specified by NF EN 1992, Article 3.1.7 (2). Whitney’s equivalent rectangular stress block has been proven since the 1960’s to be the simplest stress distribution for calculating flexural strength (Mattock et al 1961). The following discussion demonstrates its accuracy in comparison with more complex compressive stress distributions. The ultimate compressive strain at the extreme compression face of the member is still assumed here to be 0.003 for consistency with both the current ACI and the AASHTO provision. An argument can be made to use 0.0035 as advocated in Canada and Europe. However, for typical commercial production, members are generally under-reinforced, and the depth of the compression zone is generally very small compared to the full member depth. Thus, the difference in assumed ultimate strain results in essentially no difference in the flexural capacity.
In cases outside of the scope of this project, where the compression zone is found to be adequately confined to result in a higher ultimate strain, member ductility, and its ability to house more reinforcement than the conventionally used values, can significantly improve, see Aghdasi et al. (2016), Flexural behavior of prestressed concrete members is checked in design at two loading levels: (a) service limit states with loading combinations used with assumed load factors = 1.0, and (b) strength limit states with load factors and loading combinations specified in the relevant code. The discussion in this section is limited to the capacity side of the capacity-demand relationship. It is applicable whether a building or bridge product is being designed.

4.3 Experimental Stress-Strain Relationships

Flexural tension of UHPC is greatly enhanced due to the presence of high strength steel fibers. In this proposal, minimum values of the strength in flexural tension are specified from testing performed according to ASTM C1609 Standard. The load-deflection curve according to this standard is given in Figure 4.3-1 with the minimum values of the load specified at specific points on the graph for testing at concrete age = 28 days. Note that the relationship between load \( P \) and stress \( f_f \) is derived as follows. Moment, \( M = PL/6 \); stress \( f_f = M/S = 6M/bd^2 \); thus \( f_f = PL/b^2 \). For \( L = 12 \text{ in.} \) and \( b=d=4 \text{ in.} \), as specified in ASTM C1856, the load corresponding to first peak cracking stress \( f_{fc} \) of 1.50 ksi is 8.0 kip. The load corresponding to ultimate flexural strength \( f_{fu} \) of 2.00 ksi is 10.7 kip. The ACI 318-14, Section 26-12-5, provisions for ductility are adopted here. It is specified that the residual strength at deflection of \( (L/300 = 0.04 \text{ in.}) \) be at least 90% of the cracking strength and at \( (L/150 = 0.08 \text{ in.}) \) be at least 75% of the cracking strength. This requirement has been retained for the materials developed in this study.

![Figure 4.3-1. Minimum Flexural Tension Requirements](image)

In compression, standard 3 by 6-inch cylinder testing is performed. The minimum specified strength at 28 days is required to be 18.0 ksi. A typical stress-strain relationship is given in Figure 4.3-2. Note that the relationship is almost linear to the peak strength. The modulus of elasticity is assumed = 6,500 ksi as a default value unless testing is specifically performed and a better value is established.
4.4 Idealized Stress-Strain Diagrams
The actual relationships are simplified in various guidelines and codes of practice in order to allow for accelerated design without loss of safety or significant waste of materials. The following section illustrates how the relationships are simplified in various international sources. It is followed by recommendations for use in this proposal.

4.5 Previous Work
Figure 4.5-1 (Gowripalan and Gilbert, 2000) shows an idealization of the stress-strain relationship of UHPC in compression and in tension. It shows that the compressive stress-strain relationship is expressed by a trilinear diagram with the peak stress limited to 0.85 $f'_{c}$. The tensile stress-strain relationship is also expressed as a trilinear one. The stress in the diagram is not to scale, for clarity, as compression is nearly 10 times higher than tension. The researchers proposed an ultimate compression strain limit of 0.0035 at the extreme compressive fibers. Note that the descending parts of the diagrams would not be useful in structural design of actual members subject to increasing loads. These values are only obtained if the member is in a testing lab and the load is applied hydraulically with displacement control equipment.

The modulus of elasticity in tension and compression are assumed equal in the model. It is assumed = 6,500 ksi in this report. Application of this diagram to the limits specified for design would result in a peak compressive stress $= (0.85)18 = 15.30$ ksi. The values of $f_{c} = 1.50$ ksi and $\frac{5}{8} f_{c} = 0.94$ ksi result in strains at the end of the first linear segment $= 15.3/6500 = 0.00235$ in compression and $0.000144$ in tension. The strain at the end of the horizontal line is $0.004$ in compression and $0.16L_{f}/1.2h = (0.16)(0.75)/1.2h = 0.1h$, with a limit of $0.004$, where $h$ is the total member depth in inches. The total strain limit at the end of the descending line (at zero stress) is set at $0.007$ in compression and $L_{f}/1.2h = 0.625/h$, but not greater than $0.01$, in tension. As will be seen later, the stress-strain diagram proposed to be used in this report is similar to that shown in Figure 4.3-1 with the descending lines eliminated for design purposes (not for evaluation of test specimens under displacement-controlled loading). Other simplifications are given later in this section.
For concrete members reinforced exclusively with fibers in flexure, Gowripalan and Gilbert (2000) proposed that the flexural capacity may be determined based on ultimate tensile strain of $0.16L_c/1.2h$, but not to exceed $0.004$, at the extreme tensile fibers. The approach is general and is equally applicable whether continuous reinforcement is used or not. It involves plotting the moment-curvature diagram at various levels of concrete extreme fiber strain. To obtain a point on the diagram, a value of the strain of extreme compressed fiber, $\varepsilon_c$, is selected. See Figure 4.5-2. The stress distribution over the depth of the beam is dependent on the strain values at various distances from the neutral axis. Through an iterative process, a neutral axis position is tried, and the neutral axis depth is changed until the stress resultants in tension and compression are equal. The curvature of the beam is determined by dividing the compression strain, $\varepsilon_c$, by the depth of the compression zone. The moment is determined by summing up the products of the force components (tension and compression) times the distance from a fixed level in the section, say the top fiber. Gowripalan and Gilbert (2000) recommended that the strength reduction factor be taken as 0.8 and 0.7 for UHPC with and without continuous reinforcement respectively.
Graybeal (2008) evaluated the structural behavior of UHPC prestressed I-Girders. Figure 4.5-3 shows the proposed stress-strain response in tension and compression. Graybeal's simplified stress-strain response was considered conservative in comparison with flexural strength experiments. The ultimate compressive stress and strain were limited to 0.85 $f'_c$ and 0.0035, respectively. The ultimate tensile stress and strain were limited to 1.50 ksi and 0.007, respectively.

Figure 4.5-2. Strain and Stress Distribution on Rectangular Section in Pure Bending after Gowripalan and Gilbert (2000)
Vande Voort et al. (2008) investigated the performance of UHPC for deep foundations. They modeled the compression behavior as a trilinear diagram in accordance with Gowripalan and Gilbert (2000). The tension behavior was modeled according to recommendations by Bristow and Sritharan, Eqs. (4.5-1) – (4.5-4). The equations presenting the stress-strain diagram are shown below. Bristow and Sritharan also recommended a modulus of elasticity, elastic tensile strength, and maximum tensile strength of 8,000, 1.30, and 1.70 ksi, respectively, for a commercial class of UHPC used in their project. **Figure 4.5-4** presents the stress-strain behavior in tension and compression as used by the researchers to evaluate flexural performance of UHPC piles.

\[
 f_t = E_c \varepsilon 
\]

for \( \varepsilon \leq f_{tc}/E_c \)  
\[ Eq. \ 4.5-1 \]

\[
 f_t = f_{tc} + \frac{(f_{tu}-f_{tc})(\varepsilon-f_{tc}/E_c)}{0.00125} 
\]

for \( f_{tc}/E_c < \varepsilon \leq 0.0014 \)  
\[ Eq. \ 4.5-2 \]

\[
 f_t = f_{tu} 
\]

for \( 0.0014 < \varepsilon \leq 0.0024 \)  
\[ Eq. \ 4.5-3 \]

\[
 f_t = f_{tu} - 0.672 \ln(\varepsilon) - 4.062 
\]

for \( \varepsilon > 0.0024 \) until \( f_t \) reaches 0 ksi  
\[ Eq. \ 4.5-4 \]

where

- \( f_t \) = concrete tensile stress (ksi)
- \( E_c \) = modulus of elasticity of UHPC (ksi)
- \( \varepsilon \) = tensile strain (ksi)
- \( f_{tc} \) = first-crack tensile strength (ksi)
- \( f_{tu} \) = ultimate tensile strength (ksi)
Fehling and Leutbecher (2011) proposed a simplified stress-strain relation in compression and tension as shown in Figure 4.5-5. The diagram was proposed to have a triangular stress distribution with ultimate stress at the extreme fibers of 0.85 $f'_c$. The tensile stress-strain curve was simplified using a rectangular block with an average stress of 90% of the tensile strength in most cases and 85% in the case where the cross-section width reduces towards the tension side. This block is distributed over 90% of the depth of the tension zone.

Naaman (2017) performed a numerical investigation to study the effect of idealized relationships in compression and tension on the nominal flexural resistance of fiber reinforced concrete (FRC) without longitudinal reinforcement. Figure 4.5-6 shows the simplified stress distribution in compression and tension. The compression curves are modeled as parabolic, rectangular and triangular blocks regardless the fact that different FRC has the same response under compression. Unlike the compression response, the tensile response is different from one FRC to another. The post-cracking tensile behavior varies between strain-hardening and strain softening. Even for the same type of FRC, the actual stress-strain curve may or may not have another peak. Naaman modeled the tensile stress distribution as a linear diagram with a uniform value representing the average of first-crack stress and post-crack strength for strain hardening FRC. For strain softening FRC, the tensile stress distribution was modeled as a triangle with a maximum value of the post-cracking strength.

Naaman's investigation reported that the modeling of compression is insensitive to the shape of the compression block. On the other hand, the nominal moment decreased by 28% by changing the rectangular tensile block to a triangular one with the same average stress. Accordingly, the author's proposal in this report does not allow for a material with strain softening in tension. The peak (ultimate) flexural strength shall be at least 1.25 times first-peak
(first-crack) flexural strength. Thus, the fear of reduced flexural strength is alleviated in members with no continuous reinforcement when a rectangular stress block in tension is used.

**Figure 4.5-6. Typical Stress Block Models for Simplified Analysis (Naaman, 2017)**

The International Federation of Concrete (fib) Task Group 8.6 Report (2011) proposed a simplified stress-strain diagram for concrete in flexural compression. It consists of a bilinear stress-strain diagram with an initial slope of $E_c/1.3$. When the stress reaches 0.85 $f'_c$, it becomes horizontal until the strain reaches ultimate strain, assumed equal to $f'_c/E_c$. A similar bilinear diagram is specified by the French AFGC (2013) with a couple differences. First, it specifies that $E_c$ be used for the slope of the first line. Second, the ultimate compressive strain is made a function of the post-cracking stress in tension and the compressive strength. AFGC also specifies reducing the tensile strength by a factor accounting for the fiber orientation and its effect on the strength.

The Korean Concrete Institute's Guidelines for K-UHPC Structural Design (2012) specifies an ultimate compressive strain of 0.004. The relationship is assumed bilinear for UHPC with strain hardening in tension. The first line has a slope of the modulus of elasticity until reaching the first-crack tensile strength, then, the strain increases by increasing the stress to reach the post-cracking tensile strength and the ultimate strain limit.

The Japanese Society of Civil Engineers (JSCE) Draft Recommendations for Design and Construction of Ultra High Strength Fiber Reinforced Concrete Structures (2016), **Figure 4.5-7** shows the relationship between stress and strain in compression and in tension. $\epsilon_m$ and $\epsilon_{cu}$ should be determined by physical testing. The initial curve zone is described by Eq. (4.5-5). It should be noted that the partial safety factors used in some codes for certain UHPC parameters, are omitted within this report following the same philosophy used in the US codes. Rather, all the factors are considered in the strength reduction factors (resistance factors).
Figure 4.5-7. Idealized Stress-Strain Relation in Compression and Tension as Specified by JSCE (2016)

\[
f_c = 0.85 f'_c \frac{E_c}{E_m} \left(2 \frac{\varepsilon_c}{\varepsilon_m} \right)
\]

Eq. 4.5-5

Annex 8.1 of the Canadian Highway Bridge Design Code was released in 2018 for public review. This Annex specifies design guidelines for fiber reinforced concrete (FRC). Flexural component design is discussed in Article A8.1.8.4 including members without continuous reinforcement in the tensile zone. The moment curvature response, given by Gowripalan and Gilbert (2000) is adopted. Limits were specified to avoid brittle failure due to the pullout of the fibers before yielding of the longitudinal reinforcement. The curvature-ductility ratio, which is defined as the ratio of the curvature at the maximum calculated flexural resistance to the curvature at the flexural resistance corresponding to yielding of the reinforcing in the tension zone, is specified to not be less than 2.0. Otherwise, the factored flexural resistance shall be the greater of: (a) 0.67 times the maximum calculated flexural resistance including the contribution from fibers; and (b) the flexural resistance of the section based on only longitudinal reinforcement, without considering the contribution of fibers. The maximum contribution of the fiber was also limited by specifying that the minimum flexural strength contributed by the reinforcing bars be at least $= 80\%$ of the factored load demand. Eliminating the reinforcing bars is only permitted for secondary components. For these components, the factored moment demand shall not exceed 0.50 times the predicted flexural resistance, to account for the potential of brittle failure.

In this project, the essence of the Canadian guidelines is proposed to be adopted. Specifically, (1) use the larger of $M_{n1}$ or $0.8M_{n2}$, where $M_{n1}$ and $M_{n2}$ are the flexural strength with fibers ignored and with fibers considered, respectively; (2) when no longitudinal steel is used, $M_{n2}$ becomes the strength with the fibers only; for this case, an additional factor of safety is required such that $M_{n2}$ is not less than $2.0M_u$, where $M_u$ is the factored load moment demand.

### 4.6 Proposed Relationships

A tensile and compressive stress-strain relationship is required for analysis for flexure. For service load analysis, the linear stress-strain diagram assumption is generally acceptable and will be retained here. For strength analysis, the strain compatibility approach is used with idealized stress-strain diagrams as given below.
4.6.1 Service Limit State

Analysis of prestressed concrete members for service limit states, whether the analysis is done at time of prestress release or in service when full loads are applied, is straightforward. It is specified for the concrete being used in this study that the flexural cracking limit is a minimum of 1.5 ksi at concrete age of 28 days, see Figure 4.3-1.

For stress limit at Service III limit state for bridges and at the “uncracked” category service load check for buildings, it is recommended to use a conservative 1.00 ksi stress limit at the bottom fibers at the ends of the member is assumed. This is consistent with the ratio $\sqrt{10/18} = 0.745$, where 10.0 ksi is the minimum specified compressive strength at prestress release and 18.0 ksi at service.

Flexural stress analysis for non-prestressed members is not required by design codes, except for analysis for crack control and deflection control. Such analysis is not a significant component in this research project, except for the transverse direction of prestressed members with wide top flanges. This analysis will be addressed in Phase II.

4.6.2 Strength Limit State

Flexural strength of relatively large precast prestressed concrete members is dominated by the strands in tension and the top flange of the member in compression. As will be demonstrated later, the tensile capacity of the fibers below the neutral axis at full in-service conditions can be neglected for members prestressed with the minimum level of prestressing specified by the current codes. This situation covers the great majority of cases encountered for primary flexural members in buildings and bridges. However, there are some situations where it may be desirable to eliminate continuous reinforcement and to count on the fibers to resist flexure. Examples are the transverse design of ribbed deck slabs and ribbed top flanges of decked I-beams. Also, box beams and voided slabs are candidates for elimination of the transverse reinforcement, as the spans between webs are relatively small, ranging from 3.0 to 6.0 ft.

For analysis purposes, it is convenient to simplify the stress-strain diagrams in tension and in compression as shown in Figures 4.6.2-1 through 4.6.2-3. Figure 4.6.2-1 models the relationship in tension as elastic-perfectly-plastic. This is conservative as it ignores the strain hardening. The tensile strength limit of 0.75 ksi is proposed in the preceding section “Non-Linear Stress Analysis”. The capacity goes to zero at a strain of 0.004 which is more conservative than the 0.007 recommended by some researchers. These conservative assumptions are only significant in the cases where continuous reinforcement is not used, which will be shown to be impractical in major flexural members.

![Figure 4.6.2-1. Idealized Tensile Stress-Strain Diagram](image)

The idealized relationship on the compression side of the member at ultimate flexure is given in Figure 4.6.2-2. It represents the standard Whitney equivalent rectangular stress block and the standard ultimate strain of 0.003 generally used in US codes. For concrete strengths larger than 10.0 ksi (Rizkalla et al., 2007), it has been found that the stress-strain relationship is almost a straight line up to the peak strength. When that fact is considered, the $\beta$
parameter, which is the ratio between the depth of the compression block and the neutral axis, is 0.65 (approximately 2/3) and the strain at the start of the compression block can be shown to equal 0.001.

![Graph showing compression stress-strain relationship](image)

**Figure 4.6.2-2. Idealized Compression Stress-Strain Diagram for Design of Members with Continuous Reinforcement**

In situations where there is no flexural rebar or strand in the cross section, the strain in compression cannot reach the ultimate value of 0.003. In these situations, the compressive stress-strain relationship may be assumed linear as shown in **Figure 4.6.2-3**. These situations may be encountered, for example, in transverse design of decked I-beam flanges and voided box flanges.

![Graph showing linear stress-strain relationship](image)

**Figure 4.6.2-3. Idealized Compression Stress-Strain Diagram for Design of Members without Continuous Reinforcement**

### 4.7 Examples of Contribution of Fibers to Flexural Strength

#### 4.7.1 Prestressed UHPC Decked I-Beams

The decked I-beam being used as one of the recommended products of this research is used here to investigate the contributions of the various components of the section to the flexural strength of the midspan section of the member. **Figure 4.7.1-1** shows the cross section at midspan with a template of the strand positions that can be placed in the bottom flange. It is required here to determine the relationship between the area of prestressing strands provided in the bottom flange and the nominal flexural capacity of the member using the simplified stress-strain relationships recommend in the preceding section. For convenience, in a spreadsheet analysis, the section is
converted into a series of rectangles from top to bottom. The ribs in the transverse direction are assumed not to contribute to longitudinal flexural capacity. A strain-compatibility spreadsheet was used for the analysis. In it, the concrete relationships given here were programmed. Also, the Power Formula (see PCI Bridge Design Manual) was used to model the prestressing steel.

Figure 4.7.1-1. Decked I-Beam used to Evaluate the Contribution of Fibers in Prestressed Members

Figure 4.7.1-2 shows the nominal flexural strength to area of prestressing steel relationship with fibers considered, $M_{n2}$, and fibers ignored, $M_{n1}$. It is very clear from this example that in primary prestressed concrete members, contribution of the fibers to flexural strength is negligible.
Figure 4.7.1-2. Relationship between Nominal Flexural Strength and Area of Prestressing Steel for a 4ft-deep Decked I-Beam

In the event there is no strand in the member, Figure 4.7.1-3, shows the moment-curvature relationship, established using the compressive stress-strain diagram given in Figure 4.6.2-3. The proposed tensile stress-strain diagram remains the same whether longitudinal steel is provided or not. The reason for using a straight-line model for compression up to 0.85$f'_c$ is that a lack of longitudinal steel will cause the peak moment in the moment-curvature relationship to be at a relatively low strain in both tension and compression. This is reflected in Figure 4.7.1-3. This causes the design to be iterative when no longitudinal steel is provided. Compared to the values given in Figure 4.7.1-2, the peak moment is relatively small, and is further reduced due to lack of required ductility as given earlier. Figure 4.7.1-3 also shows the peak moment when the section is 8-ft deep. The conclusions remain the same. There is no need, or justification for counting on the fibers to resist any significant amount of the flexural resistance in prestressed members.

Figure 4.7.1-3. Relationship between Nominal Flexural Strength and Curvature for Decked I-Beams without Longitudinal Reinforcement

4.7.2 Conventionally Reinforced UHPC Decks

Figure 4.7.2-1 shows the moment curvature relationship for a UHPC solid deck slab in the transverse direction between beam lines. This analysis has been performed for a deck thickness of 8 in. and strip width of 48 in. The deck is reinforced with three different levels of continuous reinforcement in addition to a section without reinforcement. For sections with continuous reinforcement, maximum curvature was calculated with an ultimate concrete compression strain of 0.003. The same conclusions drawn for the prestressed beam can be made here. It is generally not recommended to use fibers only without longitudinal reinforcement, even for a thick rectangular structural member.
4.7.2-1. Relationship between Nominal Flexural Strength and Curvature for an 8” Solid Slab

4.7.3 1-in. Thick Slab
One more example is given in Figure 4.7.3-1. It shows the moment-curvature relationship for a 1-in. thick slab with no reinforcing bars provided to help resist the applied flexure. An example of such element is the transverse direction of the voided slab system proposed for buildings. The thickness was chosen to keep the weight of this joint system to a minimum. As such, no reinforcing bars are provided and only the fibers are counted on to provide the required resistance to the factored moment load of (150 psf/1000) (3)²/12 = 0.1125 kip-ft/ft. The figure shows that it is possible to achieve a moment, \( M_{n2} \), of 0.32 kip-ft per ft of length. If the design strength of \( 0.5M_{n2} \) is used, the capacity is still higher than the demand. This would be one of few examples where it may be feasible to count on the fibers only without any additional reinforcing to supply the required strength.

4.7.4 Ribbed Slab
One final example is the transverse direction of the decked I-beam. It can be shown that providing no transverse reinforcement in the top of the deck may be acceptable for relatively short spacing between beams. The skin thickness is 3 inches and the total ribbed slab thickness is 8 in. as shown in Figure 4.7.4-1. The area of the top flange of the T-section, which is in tension for negative moment near the face of the web is relatively large = (3) (24) = 72 in², where the 24 in. is the spacing between ribs. Thus, the fiber capacity in tension in the top flange is relatively large. Such a situation can be analyzed using the proposals given here and the designer can make a decision whether to accept the design without reinforcing bars or use bars as an additional redundancy in the design.
4.8 Proposed Tentative Design Recommendations

4.8.1 Sections without Continuous Reinforcing
Eliminating the continuous reinforcement is exclusive to secondary elements. To evaluate the reduced flexural capacity of these members, moment-curvature diagrams should be developed using the strain compatibility approach with the idealized stress-strain curves as proposed in Figures 4.6.2-1 and 4.6.2-3. The nominal flexural strength is the peak value of the curve. A strength reduction factor of 0.5 shall be used.

4.8.2 Sections with Continuous Reinforcing
All primary members shall be reinforced with continuous rebar and/or pretensioned strands. In the case of strand reinforcing, the members shall be checked at the service limit states and ultimate limit states. At service limit states, no cracks are allowed either at the time of releasing the strands or at final in-service conditions. At ultimate limit states, strain compatibility shall be used to calculate the nominal flexural capacity. It is proposed to use the strength reduction factors as specified by ACI318-14 or AASHTO LRFD 8th for buildings and bridges, respectively. The recommendations for flexural design of members with continuous reinforcing may be summarized as follows:

4.8.2.1 Service Limit States for Prestressed Members

4.8.2.1.1 At Release
It is proposed to check the stresses at the beam ends at the time of transfer. The tensile and compressive stresses shall not exceed the proposed limits as shown in Eqs. (4.8.2.1.1-1) and (4.8.2.1.1-2). The stresses should be checked at the transfer length, however, it is conservative to check these stresses at the end of the beams. If the stresses exceed the limits, top strands may be added or some of the bottom strands may be debonded. It is also recommended to check these stresses everywhere along the beam length.

\[
\begin{align*}
\sigma_t &= \frac{P_{pi}}{A_{ti}} - \frac{P_{pe ti}}{S_{tti}} + \frac{M_g}{S_{tti}} \leq -0.75 \text{ ksi} \quad \text{Eq. 4.8.2.1.1-1} \\
\sigma_b &= \frac{P_{pi}}{A_{ti}} - \frac{P_{pe ti}}{S_{bti}} + \frac{M_g}{S_{bti}} \leq 0.70 \sigma_c' \\
\end{align*}
\]

where
- \( \sigma_t \) = top fiber normal stresses due to prestress and external loads under the service limit state (ksi)
- \( P_{pi} \) = total prestressing force before transfer (kips)
- \( A_{ti} \) = area of transformed section at transfer (in.²)
- \( e_{ti} \) = eccentricity of strands with respect to the transformed section at transfer (in.)
- \( S_{tti} \) = section modulus for the extreme top fiber of the transformed section at transfer (in.³)
- \( M_g \) = unfactored bending moment due to the beam self-weight (ft-kips)
- \( \sigma_b \) = bottom fiber normal stresses due to prestress and external loads under the service limit state (ksi)
- \( S_{bti} \) = section modulus for the extreme bottom fiber of the transformed section at transfer (in.³)
- \( \sigma_c' \) = required concrete compressive strength at transfer (ksi)

4.8.2.1.2 At Service
It is proposed to check the stresses at the beam midspan at service limit states after all prestressing losses. The tensile and compressive stresses shall not exceed the proposed limits as shown in Eqs. (4.8.2.1.2-1) and (4.8.2.1.2-2). The stresses should be checked at all sections along the beam length; however, mid-span sections typically control. It should be noted that live load effects should be reduced by 20% for checking the tensile stresses.
of bridge beams in accordance with AASHTO LRFD load combinations. The prestressing losses calculations can be found in creep and shrinkage section.

\[
f_t = \frac{P_{po}}{A_{ti}} + \frac{P_{po}e_{ti}}{S_{tti}} - \Delta f_{plT1} A_{ps} \left( \frac{1}{A_g} + \frac{e_{pg}(y_b - h)}{I_g} \right) + \frac{M_s}{S_{ttf}} \\
\leq 0.6 f'_{ct} \quad \text{Eq. 4.8.2.1.2-1}
\]

\[
f_b = \frac{P_{po}}{A_{ti}} + \frac{P_{po}e_{ti}}{S_{bti}} - \Delta f_{plT1} A_{ps} \left( \frac{1}{A_g} + \frac{e_{pg}(y_{bc} - h)}{I_g} \right) - \frac{M_s}{S_{btf}} \\
\leq -1.00 \text{ksi} \quad \text{Eq. 4.8.2.1.2-2}
\]

where

- \( f_t \) = top fiber normal stresses due to prestress and external loads under the service limit state (ksi)
- \( P_{po} \) = total prestressing force after early-age shrinkage losses (kips)
- \( A_{ti} \) = area of transformed section at transfer (in.\(^2\))
- \( e_{ti} \) = eccentricity of strands with respect to the transformed section at transfer (in.)
- \( S_{tti} \) = section modulus for the extreme top fiber of the transformed section at transfer (in.\(^3\))
- \( S_{bti} \) = section modulus for the extreme bottom fiber of the transformed section at transfer (in.\(^3\))
- \( M_s \) = unfactored bending moment due to the beam self-weight (kip-in.)
- \( \Delta f_{plT1} \) = long-term losses due to shrinkage and creep of concrete, and relaxation of steel from transfer to casting deck (ksi)
- \( A_{ps} \) = area of bonded strands crossing the section under investigation (in.\(^2\))
- \( A_g \) = area of cross section of the noncomposite precast beam (in.\(^2\))
- \( e_{pg} \) = the distance between the center of gravity of bottom strands and the bottom concrete fiber of the beam, gross section (in.)
- \( y_b \) = distance from centroid to extreme bottom fiber of the noncomposite precast beam (in.)
- \( h \) = overall depth of beam (in.)
- \( I_g \) = moment of inertia about the centroid of the noncomposite precast beam (in.\(^4\))
- \( M_c \) = unfactored bending moment due to slab and haunch weights (kip-in.)
- \( S_{ttf} \) = section modulus for the extreme top fiber of the transformed section at final time (in.\(^3\))
- \( \Delta f_{plT2} \) = long-term losses due to shrinkage and creep of concrete, and relaxation of steel after casting deck to final (ksi)
- \( A_c \) = total area of the composite section (in.\(^2\))
- \( e_{pc} \) = eccentricity of strands with respect to the centroid of the composite section (in.)
- \( y_{bc} \) = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam (in.)
- \( I_c \) = moment of inertia of the composite section (in.\(^4\))
- \( M_{SID} \) = unfactored bending moment due to super-imposed dead loads (kip-in.)
- \( S_{ttc} \) = composite section modulus for the extreme top fiber of the precast beam for transformed section at final time (in.\(^3\))
- \( M_{LL} \) = unfactored bending moment due to live load (kip-in.)
- \( f'_{ct} \) = required concrete compressive strength at transfer (ksi)
- \( f_b \) = bottom fiber normal stresses due to prestress and external loads under the service limit state (ksi)
- \( S_{btf} \) = section modulus for the extreme bottom fiber of the transformed section at final time (in.\(^3\))
- \( S_{btc} \) = section modulus for the extreme bottom fiber of the transformed composite section at final time (in.\(^3\))

### 4.8.2.2 Ultimate Limit State

#### 4.8.2.2.1 Flexural Strength

For non-prestressed members, it is proposed to calculate the nominal flexural capacity ignoring fibers, \( M_{n1} \), and flexural capacity accounting for fibers, \( M_{n2} \). To ensure ductility, \( M_{n1} \) should not be less than 0.8\( M_u \). If not, the nominal flexural capacity is the larger of \( M_{n1} \) or 0.67\( M_{n2} \). For prestressed members, it is proposed to calculate the nominal
flexural capacity ignoring fibers. The strain compatibility approach should be used assuming the idealized stress-strain curves as proposed in Figures 4.6.2-1 through 4.6.2-3. The strength reduction factors per ACI 318-14 for buildings and AASHTO LRFD 8th for bridges should be applied. It also should be noted that there are no proposed changes to the tensile resistance from steel strands and steel rebar. However, it is highly recommended to use the power formula as presented in PCI Bridge Design Manual to calculate the average stress in prestressing steel, \( f_{ps} \).

4.8.2.2 Minimum Flexure Reinforcement

While AFGC (2013) and other guidelines have dropped the requirement for minimum flexure reinforcement, it is proposed to use current specified equations for conventional concrete by ACI 318-14 and AASHTO LRFD 8th with one change. The change is to substitute the modulus of rupture with the proposed first peak flexural strength of 1.50 ksi.

4.9 Plans for Phase II Testing

4.9.1 Bridge Decked I-Beam

It is planned to conduct a laboratory testing for a full-span prototype UHPC decked I-beam. A successfully designed for a 50-ft span bridge that utilizes the UHPC decked I-beam section is shown in Figure 4.9.1-1. This beam has 14 0.6-in. diameter straight strands in the bottom flange with no strands and no reinforcing bars at the top of the section. This beam is similar to the decked I-beam provided as a bridge design example and will be tested to evaluate the structural behavior in global (beam) and local (deck) flexure. In addition, factors such as losses, camber, end zone stresses, development length requirements, and bond capacity between the UHPC and the strands will be studied on the specimen wherever possible. It is envisioned that the test beam will be fabricated at FACCA (Precaster F) and transported to NC State University for testing.

The primary goal of these laboratory tests is to validate the prediction models proposed in preceding sections under strength and service limit states. Numerous tests are planned on the single full-scale beam. Initially, careful tracking of the camber, any end-zone cracking, and any strand slip will provide insight prior to loading the beam in the laboratory. The beam will be initially loaded to failure in flexure, likely in a four-point bending configuration. It is planned to hold a selected level of load for 24 hours prior to continuing the test to failure. Vertical load-deflection and load-strain behaviors of the cross-section will be measured, and a failure mode near the mid-span is expected. All tests would proceed incrementally toward failure with loading paused at regular increments for observation and documentation. On this beam, as well as the other full beam tests, ultimate compression flexural strain will be measured and compared with the proposed values of 0.003 as well as higher values reported in other codes and research reports.

After failing the beam in flexure on its full span, the supports will be reconfigured to a shorter span that will enable testing each end of the beam to failure in shear. It is anticipated that the beam ends will be largely undamaged from the previous flexure failure, and that two shear tests to failure can be obtained (one at each end), see section 7.8. After shear testing, it is hoped that sufficient areas of largely-undamaged deck will remain to enable locally loading the deck itself. This will enable to test the two-way flexural behavior of the deck top skin. After testing, the beam will be saw-cut to visually inspect the fiber distribution in the cross section. Saw-cutting will also facilitate disposal. It is also possible that the beam will have to be cut after the initial flexure test to more easily enable shear testing.
Flexural Design

Figure 4.9.1-1. Decked I-Beam Section for a 50-ft Bridge

Transverse deck slab flexure (negative moment behavior) will be studied with the test setup illustrated in Figure 4.9.1-2. As mentioned earlier in this section, such tests are important to resolve the complications in flexural theory that exist when concrete contains a large quantity of reinforcement in addition to discrete reinforcing bars. Results will enable assessing the need for transverse rebar, minimum reinforcement requirements, and the necessary resistance factors (i.e. based on failure type). This test will validate the proposed design recommendations to include the fiber contribution in flexural design.

Figure 4.9.1-2. Deck Flexural Strength Test Setup

The flexural behavior of the top skin of the ribbed slab, top flange of decked I-beam, will be investigate as shown in Figure 4.9.1-3. This is considered as two-way bending behavior of the top skin. It was proposed to reduce the top skin from 3 in. to 2 in. where the punching strength of the 2 in. skin exceeds the demand. It should be noted that the top skin does not have any rebars. The flexural tensile strength is achieved by fiber contribution. The ultimate moment at failure will be compared with that predicted using a moment-curvature diagram. The effective width of the strip will be determined analytically using finite element software.
4.9.2 NEXT Bridge Beam
It is also planned to test a modified Northeast Extreme Tee (NEXT) Beam in flexure and shear. This product is modified for implementation of UHPC in bridge construction. The top flange is a ribbed slab similar to that of the decked I-beam as shown in Figure 4.9.2-1. A 60-ft beam will be tested by NCSU with a similar testing procedure to the decked I-beam. The same beam may also be tested to evaluate the shear behavior at both ends. Only one end will have anchorage improvement, by welding rebar to the end plate.

4.9.3 Building Voided Slab
The research team has developed a UHPC voided slab supported by the UHPC inverted tees as show in the following product development section. This will be an elegant system with significant potential. This system has garnered interest from several producer partners. Initial calculations and forming designs indicate that the system will be effective, but prototypes and tests are needed. This will provide a ready-to-use UHPC floor system. The test setup of 60-ft long voided slab is shown in Figure 4.9.3-1. The specimen will be cast in two pieces and connected at the mid-height with sleeves and bars. The sleeves will be filled with UHPC and cured before shipping to NCSU.
4.10 Pilot Testing: Building Voided Slab

One UHPC voided slab was tested in this pilot program. The specimen was 60 feet long, 3’-10” wide, and 1’-10” deep. The cross section of the voided slab is shown in Figure 4.10-1. The Specimen was fabricated from (4) separate pieces: (1) continuous 60’ bottom element consisting of the stems and bottom slab, and (3) separate flat 1” thick by 20’ long lids making up the top slab. A drywall joint compound was placed at the interface between the topping elements to the bottom element. Vertical UHPC grout pockets were used to connect the topping elements to the stems without any rebars crossing the interface plane. The specimen had (16) transverse voids (8 per stem) that were spaced evenly about the mid-span. Voids were held back from the end regions of L/4. One end was reinforced for bursting resistance, confinement, and anchorage enhancement. The other end does not have any rebars. No visible cracks were noticed at either end after strands were released.

Schematics of the two flexural test setups used are shown in Figures 4.10-2 and 4.10-3. A photo of one of the setups is shown in Figure 4.10-4. The ends of the specimen were supported directly on 2” diameter solid steel rollers that rested on steel supporting beams, in turn resting on the laboratory floor. For the shear tests, the end closest to the load was supported on a 4” wide by ¾” thick neoprene pad spanning the width of the specimen. This pad rested on a steel supporting beam that in turn rested on the laboratory floor. The load was applied to the specimen using (2) loading beams with (2) hydraulic jacks per beam. Each loading beam was placed on top of (2) plywood bearing pads located directly over the stems. All hydraulic jacks shared the same pressure source to ensure a uniform application of load. The jacks were operated manually with an electric pump that enabled incremental application of the load.
Figure 4.10-2. Schematic of the First Flexural Test Setup

Figure 4.10-3. Schematic of the Second Flexural Test Setup

Figure 4.10-4. Setup for the Second Flexural Test

Applied loads and specimen deflections were measured for each test. Load cells mounted in the force path of the hydraulic jacks measured the applied load for all specimens. String potentiometers measured the vertical deflection of each stem underneath each load point and at the mid-span. Linear potentiometers were used to measure vertical and lateral relative displacements between the lid slab and the stems.
Specimens were loaded to a moment (including self-weight) corresponding to that which would be generated by an equivalent uniform load applied to a 60’ long by 6’ wide voided slab. The 6’ width is equivalent to the expected in-use tributary width, including flange cantilevers that did not exist on the prototype specimen. The specimen was loaded incrementally to the selected levels of superimposed distributed load shown in **Table 4.10-1**. The applied load was held at each level for the short period of time necessary to examine the specimen and to document/photograph any observations (typically less than 5 minutes at each load point).

**Table 4.10-1. Loading Sequence for Flexural Tests**

<table>
<thead>
<tr>
<th>Load Step</th>
<th>Superimposed Distributed Load (psf)</th>
<th>Bending Moment including Self-Weight (kip-ft)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>308</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>80</td>
<td>387</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>440</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>120</td>
<td>493</td>
<td>Approximate service load</td>
</tr>
<tr>
<td>5</td>
<td>140</td>
<td>546</td>
<td>Peak load applied in test 1</td>
</tr>
<tr>
<td>6</td>
<td>160</td>
<td>599</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>180</td>
<td>651</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>200</td>
<td>704</td>
<td>Peak load applied in test 2</td>
</tr>
</tbody>
</table>

A pair of 3” by 6” nominal size UHPC cylinders labeled “top pieces” were delivered with the test beam. Compression tests were performed to determine the concrete compressive strength within 3 days of beam testing. Cylinders were tested in accordance with ASTM C39 and presented average compressive strength of 16.40 ksi. Concrete first peak flexural tensile strength using ASTM C1609 standard testing averaged 2.1 ksi.

The specimen was loaded in the increments outlined in **Table 4.10-1**. The first test was terminated at a moment equivalent to a superimposed load of 140 psf – a level well beyond that seen in a typical office building. No bottom flexural cracks at mid-span were observed during this test. No relative displacement was detected between the lid and the stem in either the vertical or lateral direction. No signs of strand slip were observed. The stiffness of the element was observed to be highly linear. The mid-span deflection was measured as 1.8 in., which is more than the predicted value of 1.5 in.

The second test was conducted by loading the specimen at the same increments designated in **Table 4.10-1**. The test was terminated at bending moment of 704 kip-ft, which is equivalent to a 200 psf applied load due to relative displacement between the lid and the stem. **Figure 4.10-5** shows the transverse joint of the lid slab where the significant relative displacement occurred. No bottom visible flexural cracks at mid-span were observed during this test. No signs of strand slip were observed. The cracking moment of the composite section is predicted to be 661 kip-ft. The section with and without lid has a predicted flexural strength of 1125 kip-ft and 1013 kip-ft, respectively.
It should be noted that the proposed cross section for market production is little different than what was tested in Phase I. The proposed product will also be cast in two stages with a shape of double tees and connected with a longitudinal connection at the mid height. The Phase I prototype is different because of the production limitation at the time of trial batching. Steel forms have been ordered for the proposed final product as shown in the following section for product development. The connection of the final product will be able to develop fully composite action achieving the predicted flexural strength before any relative displacement between the top and the bottom parts.

4.11 References


ACI Committee 318. (2014). *Building Code Requirements for Structural Concrete and Commentary*, (ACI 318-14), American Concrete Institute: Farmington Hills, MI, p. 503.


Implementation of UHPC in Long-Span Precast Pretensioned Elements

Flexural Design


Japan Society of Civil Engineers (JSCE). (2006). Recommendations for design and construction of ultra-high strength fiber reinforced concrete structures (Draft), Japan Society of Civil Engineers for Concrete.


5 TIME DEPENDENT EFFECT

5.1 Notation

\( A_{ps} \) = area of prestressing steel (in.²)
\( A_c \) = area of concrete (in.²)
\( E_c \) = modulus of elasticity of concrete at 28 days (ksi)
\( E_d \) = modulus of elasticity of concrete at transfer (ksi)
\( E_{st} \) = modulus of elasticity of concrete at transfer or time of load application (ksi)
\( E_p \) = modulus of elasticity of steel reinforcement (ksi)
\( f(t-t_o) \) = kinetics creep function
\( f_\sigma \) = required concrete compressive strength at transfer (ksi)
\( f_{gap} \) = concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)
\( h \) = beam height (in.)
\( h_0 \) = prism height of standard test for length change of hardened concrete (in.)
\( h(t_o) \) = creep function
\( J(t, t_o) \) = specific basic creep between certain age and loading time (days)
\( K \) = creep coefficient
\( K_{id} \) = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for the time period between transfer and deck placement
\( k(t_o) \) = factor based on loading time, \( t_o \)
\( k_f \) = factor for the effect of concrete strength
\( k_{hc} \) = humidity factor for creep
\( k_{hs} \) = humidity factor for shrinkage
\( k_s \) = factor for the effect of the volume-to-surface ratio of the component
\( S_t \) = shrinkage after a certain time (microstrain)
\( S_{udt} \) = ultimate shrinkage (microstrain)
\( t \) = age of concrete, hours; age of concrete (days)
\( t_i \) = one-hour age, hours; one day age of concrete (days)
\( t_o \) = time infinity, all days beyond the 28-day age (days)
\( t_e \) = effective time since cast (hours)
\( t_o \) = loading time (days)
\( t_s \) = age of concrete when drying shrinkage and expansion start (days)
\( w/b \) = water-to-binder ratio
\( \alpha \) = thermal expansion coefficient (in./in./°F)
\( \beta_i \) = development coefficient over time of the drying shrinkage after high temperature moisture curing; development coefficient over time of the autogenous shrinkage
\( \beta \) = time development coefficient over time of the autogenous shrinkage of untreated UHPC
\( \Delta f_{CR+SH} \) = prestress loss due to creep and shrinkage (ksi)
\( \Delta f_{ES} \) = prestress loss due to elastic shortening (ksi)
\( \Delta f_i \) = initial prestress losses (ksi)
\( \Delta f_{SH} \) = prestress losses due to the shrinkage of UHPC (ksi)
\( \Delta T \) = change in temperature (°F)
\( \varepsilon_{aut0} \) = strain due to autogenous shrinkage of untreated UHPC (microstrain)
\( \varepsilon_{cr} \) = strain due to creep (microstrain)
\( \varepsilon_{ef(t)} \) = strain that accounts for the time development of the autogenous shrinkage of untreated UHPC (microstrain)
\( \varepsilon_{shb} \) = total shrinkage strain (microstrain)
\( \varepsilon_{shi} \) = shrinkage that happens between the time of concrete placement and the end of curing (microstrain)
\( \varepsilon_{shLT} \) = shrinkage that takes place between the end of curing and time infinity (microstrain)
\( \varepsilon_{usu0} \) = ultimate shrinkage (microstrain)
\( \varepsilon_{us(t)} \) = total shrinkage over time, \( t \) (microstrain)
\( \varepsilon_s \) = longitudinal strain (microstrain)
5.2 Introduction

The structural behavior of concrete elements is highly dependent on the time dependent effect due to shrinkage and creep. Shrinkage is the decrease in either length or volume of material, resulting from changes in moisture content or chemical changes (ACI CT-13). According to ACI 209R-92, this includes drying shrinkage, autogenous shrinkage, and carbonation shrinkage. Drying shrinkage is due to moisture loss in the concrete. Autogenous shrinkage is caused by the hydration of the cement. Carbonation shrinkage results as the cementitious materials are carbonated in the presence of carbon monoxide. ASTM C157 provides a standard procedure to determine the unrestrained shrinkage for hardened concrete by measuring the change in the length of the specimen. Recording the change in length typically starts multiple days after the concrete sets. Therefore, some of the early age autogenous shrinkage is not accounted for.

UHPC compositions exhibit greater early-age shrinkage than conventional concrete (CC). The low water-to-binder ratio of UHPC results in insufficient internal moisture to hydrate the reactive particles, resulting in high autogenous shrinkage. In turn, the prediction models for CC shrinkage are not applicable to UHPC.

Creep is defined as time-dependent deformation due to a sustained load (ACI CT-13). Basic creep is that which occurs without migration of moisture to or from the concrete. Specific creep and/or a creep coefficient may be used to evaluate the concrete creep behavior. Specific creep is defined as creep strain per unit of applied stress. Creep coefficient is defined as the ratio of creep strain to elastic strain.

5.3 Previous Work and International Practices

5.3.1 Shrinkage and Creep

Holt (2005) reported that concrete with a water-to-binder ratio of 0.45 or more does not experience autogenous shrinkage; formulations with low water-to-binder ratios – like UHPC – exhibit more autogenous shrinkage. The presence of silica fume also increases autogenous shrinkage, especially with a low water-to-binder ratio. Zhang and Leow (2003) reported that concrete with cement that has 10% of it replaced with silica fume, and a water-to-binder ratio of 0.23, experienced autogenous shrinkage three times more than the same concrete without silica fume. They also reported that most of the total shrinkage of concrete with very low water-to-binder ratio and silica fume is attributed to the autogenous shrinkage, not the drying shrinkage. Silica fume increases both drying and autogenous shrinkage, especially at the early ages (Zhang and Leow, 2003, Rao, 2001). Aïtcin (2003) and Wong (2007) reported that the carbonation shrinkage is low for high performance concrete. No carbonation shrinkage was reported for high-performance concrete with a water-to-binder ratio less than 0.3 and a silica fume content of 10% of the cement weight (Persson B., 1998). Using low amount of steel fibers in concrete results in a denser matrix, which in turn reduces the autogenous shrinkage. However, high content of steel fiber may have an adverse effect on the concrete shrinkage (Lin, 2011).

Graybeal (2006a) conducted long-term, free shrinkage testing over one year, in accordance with a modified ASTM C157. Three 3 by 3 by 11-in. prisms were cast and cured with four different regimes. The first, and most recommended regime, is steam curing, starting within 4 hours after demolding. Steam temperature was increased gradually within the first 2 hours to reach 194°F (90°C) and 95% relative humidity. The application of this high temperature steam was kept constant for 44 hours. It was then gradually decreased to room temperature over the course of an extra 2 hours. The second regime is tempered steam curing, which is similar to the first one, but with an ultimate temperature of 140°F (60°C). The third regime is delayed steam curing, which is similar to the first regime, but the steam curing is delayed by 15 days. The fourth regime is untreated curing, accomplished by setting the specimens in an ambient lab environment. Figure 5.3.1-1 shows the relation between elapsed time after demolding and the shrinkage strain. The ultimate long-term shrinkage was in the range of 766 microstrain for steam-cured prisms, and 555 microstrain for untreated prisms. These values consider the initial reading after the concrete sets. The research reported that there was no noticeable free shrinkage taking place after the end of the
steam curing process, which is in agreement with AFGC (2012) and many other researchers. For the untreated prisms, the prisms reached about 95% of the ultimate shrinkage after 60 days from the demolding date. Graybeal (2006a) proposed Eq. 5.3.1-1 to predict the shrinkage strain development at a certain age, as a function of the ultimate shrinkage strain.

$$S_t = \frac{e}{35+t} S_{ult}$$

Eq. 5.3.1-1

The researcher (Graybeal, 2006a) placed an embedded strain gage in the center of standard prisms to account for the early-age unrestrained shrinkage that occurs prior to when the concrete sets. Steam curing and the untreated curing regimes were used. The steam curing was started at an age of 1 day. The total shrinkage at an age of 90 days from casting, was reported as 850 and 790 microstrain for steam cured and untreated prisms, respectively. For the untreated prisms, the prisms reached about 50% and 65% of the ultimate shrinkage after 1 and 3 days after cast, respectively. Figure 5.3.1-2 shows the effect of the curing regime on the early age shrinkage.

Vande Voort et al. (2008) reported a literature review comparing the overall shrinkage for untreated UHPC, without accounting for the early-age shrinkage. Four previous research efforts showed an average total shrinkage of 550 microstrain, after 90 days from cast, as shown in Figure 5.3.1-3. These values are relative to the base reading at 24
hours after cast. The difference of the shrinkage strain after 7-days from demolding may be attributed to the difference of the curing regime.

**Figure 5.3.1-3. Summary of Ultimate Shrinkage for UHPC (Vande Voort et al. 2008)**

Kim et al. (2012) evaluated the effect of expansive admixture (EA) and shrinkage reducer agent (SRA) on the shrinkage strain during the early age of UHPC. The reference UHPC mixture, Mix. A, did not have any of those admixtures. Mix. B had EA and SRA of 1% and 7.5% by weight, respectively. To evaluate the shrinkage behavior before the initial setting, the setting time test was conducted. The initial setting time was 11 and 7.5 hours for the reference mix and the mix with admixtures, respectively. The total free shrinkage was recorded, including the pre-initial setting stage. The pre-initial setting part of the shrinkage strain was recorded as 250 microstrain. The total free shrinkage of the reference mixture was 800 microstrain after 91 days from casting. The EA and SRA reduced the total free shrinkage to 50% of that of Mix. A.

Budelmann and Ewert (2012) experimentally investigated the mechanical properties of UHPC at early age, including autogenous shrinkage. Two UHPC mixtures were developed. One mixture included fine grain aggregate with a maximum particle size of 0.5 mm. The other one included crushed basalt aggregate with a maximum particle size of 8 mm. The water-to-binder ratio was 0.19 and 0.21 for UHPC with fine grain and crushed basalt aggregate, respectively. The autogenous shrinkage was recorded once the concrete was placed. The specimens were stored in three different temperatures of 68 °F, 86 °F, and 104 °F. The research team evaluated an autogenous shrinkage prediction model developed by Gutsch (1998). This model, Eq. 5.3.1-2, is influenced by the water-to-binder ratio, \( w/b \). The predicted autogenous strain was 669 and 623 microstrain for UHPC with fine and coarse grain, respectively. The predicted values were in good agreement with the measured values. It was reported that there were no noticeable changes in autogenous shrinkage after 28 days from casting. Eq. 5.3.1-3 represents the time development coefficient over time of the autogenous shrinkage of untreated UHPC.

\[
\varepsilon_{a,0} = 1.3 \exp\left(-3.5 \frac{w}{b}\right) \quad \text{in microstrain} \quad \text{Eq. 5.3.1-2}
\]

\[
\beta_t = 1 - \exp\left[-0.05 \left(\frac{t_e - t_{eo}}{t_1}\right)^{0.697}\right] \quad \text{Eq. 5.3.1-3}
\]

where

\( t_e - t_{eo} \) = the effective age (hours)

\( t_1 \) = one-hour age (hours)

All the international specifications and guidelines for UHPC agree that the shrinkage after high-temperature-steam curing is not noticeable. However, the total shrinkage strain for untreated UHPC is in range of 650-950 microstrain. AFGC (2013) recommends use of 700 and 550 microstrain of shrinkage for untreated and 145 °F (65 °C) steam-cured UHPC, respectively, for the preliminary design phase. The shrinkage strain of untreated UHPC includes 150 and 550 microstrain for drying and autogenous shrinkage, respectively. It is recommended to use 550 and 0 microstrain of shrinkage for, just before and after, the end of the high-temperature-steam curing, respectively. The
high temperature is defined as 190 °F (90 °C) with a moisture content close to saturation, that is applied right after the concrete sets. Eq. 5.3.1-4 accounts for the time development of the autogenous shrinkage of untreated UHPC, where “t” is the time after setting, in days.

\[ \varepsilon_{\tau}(t) = 525e^{\left[-\frac{2.5}{t-0.3}\right]} \] (microstrain) \hspace{1cm} \text{Eq. 5.3.1-4}

While JSCE (2008) specifies determining the shrinkage of UHPC based on experiments that take into account the curing regime, JSCE (2006) recommends an autogenous shrinkage strain of 550 and 80 microstrain for untreated and steam cured UHPC, respectively. Gowripalan and Gilbert (2000) developed design guidelines for prestressed concrete beams using UHPC in Australia. They recommended an autogenous shrinkage strain of 500 and 0 microstrain for untreated and steam cured UHPC, respectively. DAFStb (2008) recommended an autogenous shrinkage strain of 600-900 and 0 microstrain for untreated and steam cured UHPC, respectively.

MCS-EPFL (2016) specified Eq. 5.3.1-5 to predict the total shrinkage strain for untreated UHPC at any age “t,” in days. The parameters of this equation are highly dependent on the cement type. The ultimate strain, \( \varepsilon_{US} \), may be assumed to be 600-800 and 950 microstrain for Portland Cement Type I and Type III, respectively. The c coefficient is -2.48 and -1.30 for Cement Type I and Type III, respectively. The d coefficient is -0.86, regardless of the type of cement.

\[ \varepsilon_{US}(t) = \varepsilon_{US\infty} e^{\left[-\frac{c}{t+d}\right]} \] (microstrain) \hspace{1cm} \text{Eq. 5.3.1-5}

Korea Concrete Institute has developed guidelines, KCI-M-12-003 (2012), for structural design that utilizes UHPC. These guidelines include two different prediction models for the development of drying and autogenous shrinkage. The ultimate shrinkage strain may be assumed as 80-100 and 450-500 microstrain for drying and autogenous shrinkage, respectively. Eq. 5.3.1-6 predicts the development coefficient over time of the drying shrinkage after high temperature moisture curing. Eq. 5.3.1-7 expresses the development coefficient over time of the autogenous shrinkage. The total shrinkage over time is the sum of the drying and autogenous shrinkage.

\[ \beta_s(t) = \left[ \frac{(t-t_s)/t_1}{350\left(h/h_0\right)^2 + (t-t_s)/t_1} \right]^{0.2} \] \hspace{1cm} \text{Eq. 5.3.1-6}

\[ \beta_s(t) = 1 - e^{\left[-0.7\left(\frac{t}{t_1}\right)^{0.5}\right]} \] \hspace{1cm} \text{Eq. 5.3.1-7}

where

- \( t \) = age of concrete (days)
- \( t_s \) = age of concrete when drying shrinkage and expansion start (days)
- \( t_1 \) = age of one day (days)
- \( h_0 \) = 3.94 in.
- \( h \) = beam height (in.)

Loukili et al. (1998) conducted a research study to evaluate the basic creep of ultra-high strength concrete with compressive strength of 23.3 to 29 ksi, at 28 days. The autogenous shrinkage was also reported. The experimental creep results were compared with the predicted model, specified by the French BPEL 91 code. The specimens were subjected to 20% of the concrete’s compressive strength at the time of loading. The creep specimens were ambient treated and loaded at different ages of 1, 4, 7, and 28 days. It was reported that the basic creep is strongly affected by the age of the concrete at the time of loading, as shown in the following figure. The creep coefficient, \( \psi \), was reported as 2.27, 1.8, 1.57, 1.08 for specimens loaded at 1, 4, 7, and 28-day age, respectively. After the heat treatment of 90 °C, the shrinkage becomes unnoticeable, and the basic creep was significantly reduced. This research was used in particular for the development of the French recommendations on UHPFRC.

Graybeal (2006a) conducted a long-term creep test over one year, in accordance with ASTM C512. Four 4-in. by 8-in. cylinders were cast and cured with four different regimes, as described earlier in this section. Figure 5.3.1-4 shows the relation between creep strain and elapsed time under load. The creep coefficient was in range of 0.29, for steam cured cylinders, and 0.78, for untreated cylinders. It should be noted that the load was applied after 4 and 28 days for steamed and untreated cylinders, respectively. The stress to strength ratio was 0.41 and 0.67 for
steamed and untreated cylinders, respectively. The creep coefficient, after 30 minutes, for pre-stressed, immature concrete with a compressive strength of 9.4 ksi and 0.6-in. strands, had a stress-strength ratio of 0.42. This value is almost half of the value for untreated UHPC after one year.

Figure 5.3.1-4. Development of UHPC Creep with Time for Different Curing Regimes (Graybeal, 2006a)

Terzijski (2008) has investigated the creep and free shrinkage of UHPC mixtures with different fiber contents. The specimens were water cured for 5 and 28 days for shrinkage and creep tests, respectively. The total free shrinkage measurements started after 24 hours from casting time. The creep specimens were loaded after 28 days, by 40% of the ultimate compressive strength. Figure 5.3.1-5 shows the relation between shrinkage and creep strain, with the elapsed time after casting. It shows the results for three sets of specimens with different steel fiber contents. The research reported that the fiber content did not have a significant effect on shrinkage or creep strain. The figure shows the total creep, including the basic and drying creep. The base creep is the difference between the shrinkage strain and the total creep strain. This shows that the average free shrinkage and creep coefficient after 300 days, are 660 microstrain and 1.5, respectively. It should be noted that the total shrinkage strain does not include the autogenous shrinkage before the initial set.

Figure 5.3.1-5. Shrinkage and Creep of Different UHPC Mixtures (Terzijski, 2008)

Flietstra et al. (2012) conducted creep evaluation of UHPC under different curing regimes to stimulate the common practice of US precast plants. These regimes are listed in Table 5.3.1-1. This research may be considered as one-of-a-kind, applying steam curing on green concrete plus applying the load on unmatured concrete. The creep test loading was applied to the concrete upon reaching 14 ksi, compressive strength. Two level of stresses, 0.2\(f'_c\) and 0.6\(f'_c\), were applied to the specimens. This research focused on the creep over 28 days. Contrary to other research efforts, the creep coefficient of steam cured specimens was higher than those of the ambient-cured specimens. This could be attributed to the steam curing accelerating the process of thinning the water layers within the concrete. Average 28-day creep coefficients of 1.12 and 0.78 were reported for steam cured specimens subjected to high and
low sustained compressive stress, respectively. They showed that creep was “locked in,” and did not change with time, for thermally treated UHPC specimens. For each of the steps following the thermal treatment, the concrete experiences no additional creep strain. The 28-day creep coefficients of 0.83 and 0.54 were reported for ambient cured specimens subjected to high and low sustained compressive stress, respectively.

**Table 5.3.1-1. Curing Regimes of UHPC (Flietstra et al., 2012)**

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description of Curing Regime</th>
</tr>
</thead>
<tbody>
<tr>
<td>AMC</td>
<td>Ambient cure for 70 hrs, then loaded in compression, continue ambient cure</td>
</tr>
<tr>
<td>SST</td>
<td>Ambient cure for 70 hrs, loaded, standard thermal cure (194 °F) was applied</td>
</tr>
<tr>
<td>PST</td>
<td>Pre-steam cure for 14 hrs, loaded, standard thermal cure (194 °F) was applied</td>
</tr>
<tr>
<td>PSD</td>
<td>Pre-steam cure for 14 hrs, loaded, ambient conditions for 72 hrs, standard thermal cure (194 °F) was applied</td>
</tr>
<tr>
<td>PDD</td>
<td>Pre-steam cure for 14 hrs, loaded, ambient conditions for 11 days, standard thermal cure (194 °F) was applied</td>
</tr>
</tbody>
</table>

All the international specifications and guidelines for UHPC agree that the high-temperature steam curing significantly reduces the creep of UHPC. AFGC (2013) recommends use of 0.8, 0.4, and 0.2 for the final creep coefficient of untreated, 145 °F (65 °C) steam-cured UHPC, and 190 °F (90 °C) steam-cured UHPC, respectively, for the preliminary design phase. Eq. 5.3.1-8 is proposed to predict the creep coefficient, accounting for the time development of the creep of untreated UHPC, where “t” is the age after loading time, $t_o$, in days. Eq. 5.3.1-9 expresses the development coefficient over time of the creep coefficient of heat-treated UHPC, where “t” is the age after loading time, $t_o$, in days.

\[
\Psi(t,t_0) = E_c f(t,t_0)
\]

where

\[
E_c = \text{modulus of elasticity at 28 days (ksi)}
\]

\[
f(t,t_0) = k(t_o) f(t-t_o) + h(t_o)
\]

where

\[
k(t_o) = 19 e^{\frac{0.1}{\sqrt{t_o - 2.65}}}
\]

\[
f(t-t_o) = \frac{t-t_o}{\sqrt{3t_o - 5}} + 1
\]

\[
(t-t_o)^{0.6}
\]

\[
h(t_o) = \frac{0.2}{18e^{\sqrt{t_o^{2} + 1.2}}}
\]

While JSCE (2008) specifies the determination of the creep coefficient of UHPC based on experiments that take into account the curing regime, JSCE (2006) recommends a creep coefficient of 1.2 and 0.4 for untreated and steam cured UHPC, respectively. Gowripalan and Gilbert (2000) recommended a creep coefficient of 1.8 and 1.2 for untreated UHPC with an applied load at 4-day age and 28-day age, respectively. They also recommended a creep coefficient of 0.5 and 0.3 for steamed UHPC with an applied load at 4-day age and 28-day age, respectively. DAfStb (2008) recommends a creep coefficient of 0.6-1.4 and 0.2-0.4 for untreated and steam cured UHPC, respectively. MCS-EPFL (2016) specifies Eq. 5.3.1-10 to predict the creep coefficient of UHPC at any age, $t$, in days. The coefficients of this equation are highly dependent on the curing regime and time of applying the load. **Table 5.3.1-2** summarizes these coefficients. KCI-M-12-003 (2012) recommends a creep coefficient of 0.45 for K-UHPC made with standard mix proportions and high temperature moisture curing, if tests are not performed.
\[ \psi(t - t_o) = \psi(t_{\infty} - t_o) \frac{(t - t_o)^a}{(t - t_o)^a + b} \]  
\[ \text{Eq. 5.3.1-10} \]

**Table 5.3.1-2. Final Creep Coefficient and Coefficients a and b (MCS-EPFL, 2016)**

<table>
<thead>
<tr>
<th>( t_o ) (days)</th>
<th>Curing</th>
<th>( \psi(t_{\infty} - t_o) )</th>
<th>a</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>68 °F (20 °C)</td>
<td>1.2</td>
<td>0.6</td>
<td>3.2</td>
</tr>
<tr>
<td>7</td>
<td>68 °F (20 °C)</td>
<td>1.0</td>
<td>0.6</td>
<td>4.5</td>
</tr>
<tr>
<td>28</td>
<td>68 °F (20 °C)</td>
<td>0.9</td>
<td>0.6</td>
<td>10</td>
</tr>
<tr>
<td>-</td>
<td>Thermal treatment - 2 days at 90 °C and steamed</td>
<td>0.3</td>
<td>0.6</td>
<td>10</td>
</tr>
</tbody>
</table>

### 5.3.2 Prestress Loss and Camber Growth

Research was conducted by Iowa State University (Bierwagen and Abu-Hawash, 2005) as a part of an Iowa DOT grant. 71-foot-long I-beams were tested to evaluate the flexural and shear capacity of the UHPC beams. The beams were a modified version of the Iowa 45-inch bulb tee section. The final measured losses were 27% higher than the predicted. The growth in camber within the period of curing was 200%, but no more growth was noted at final release. Graybeal (2006a) and Wipf et al. (2009) designed a bridge using ultra-high-performance concrete and PI girders in Buchanan County, Iowa. The losses were calculated in accordance with AASHTO LRFD 2004, and were found to be in agreement with the proposed approach.

The total losses were calculated by comparing the predicted cracking load with the measured cracking load. The creep coefficient and free shrinkage were assumed as 550 and 10 ksi at release and final, respectively. The time-dependent strain should be added to the total prestressing steel strain. The total losses of 25% of the initial stressing were calculated in accordance with AASHTO LRFD 2002, and were found to be in agreement with the proposed approach.

\[ \Delta_f = \frac{\varepsilon_{sh} A_c E_p}{E_c + A_{ps} E_p} \]  
\[ \text{Eq. 5.3.2-1} \]

Graybeal (2006b) performed design and structural testing for AASHTO Type II prestressed girders. The strands were only stressed to 55% of the ultimate strand strength. This introduced a compressive stress equal to 30% of the concrete's compressive strength, at time of release. The time of release was 3 days after casting, and steam curing followed. The time dependent losses due to creep and shrinkage were calculated using fundamental calculations. The creep coefficient and free shrinkage was determined by associated research (Graybeal 2006a).

The creep coefficient of UHPC, for this girder, was prorated to account for the level of prestressing. This gives a compressive stress that is lower than the 40% of the concrete compressive strength, at time of release, given by ASTM C512. Conservatively, the measured creep coefficient, in accordance with ASTM C512, was considered as a value between steam treated and untreated UHPC. The elastic shortening due to shrinkage that occurs after releasing the strands was calculated to account for a shrinkage strain of 350 microstrain. The modulus of elasticity was assumed to be 5,000 and 6,000 ksi at release and final, respectively. The time-dependent losses due to strand relaxation were calculated in accordance with AASHTO LRFD 2002. The losses were reported as a percentage of the jacking stress, of 10 and 14% for instantaneous and time dependent losses, respectively. It should be noted that these values correspond to a jacking stress of only 55% of the ultimate strand strength.

Wipf et al. (2009) designed a bridge using ultra-high-performance concrete and PI girders in Buchanan County, Iowa. The losses were calculated in accordance with AASHTO LRFD 1998, which was validated for UHPC, by Degen (2006). Eq. 5.3.2-2 determines the final stresses due to prestress after losses, \( \sigma_f \). The total shrinkage strain and creep coefficient were assumed as 550 microstrain and 0.3, respectively, for steam curing, per Graybeal (2006a). The predicted losses of 30% were found to be in agreement with the calculated losses from the test results. The experimental losses were calculated by comparing the predicted cracking load with the measured cracking load.

\[ \sigma_f = \frac{\varepsilon_x - \varepsilon_{sh} - \alpha \Delta T + \sigma_c \left( \frac{1}{2E_c} - \frac{K}{2E_{cl}} - \frac{1}{2E_{cl}} \right)}{\frac{1}{2E_c} - \frac{K}{2E_{cl}} - \frac{1}{2E_{cl}}} \]  
\[ \text{Eq. 5.3.2-2} \]

where

\( \varepsilon_x \) = longitudinal strain (microstrain)
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\[ \varepsilon_{sh} = \text{total shrinkage strain (microstrain)} \]
\[ \alpha = \text{thermal expansion coefficient (in./in./°F)} \]
\[ \Delta T = \text{change in temperature (°F)} \]
\[ \sigma_x = \text{longitudinal stress (ksi)} \]
\[ E_c = \text{modulus of elasticity of concrete (ksi)} \]
\[ ECI = \text{initial modulus of elasticity of concrete (ksi)} \]
\[ K = \text{creep coefficient} \]

Fehling et al. (2008) reported that the effect of high autogenous shrinkage and creep on prestress losses could be reduced significantly by applying heat curing of 194 °F (90 °C) before releasing the strands. The creep coefficient is decreased dramatically from 0.8-1.2 to 0.2-0.3, with very low residual shrinkage. The results of this research were used to design the first hybrid UHPC-steel bridge across the River Fulda in Kassel, Germany. Matties et al (2008) designed a bridge including UHPFRC components using adjacent inverted tee sections made composite with the deck under super-imposed loads. The span-to-depth ratio of the composite section was 31. The total shrinkage strain and creep coefficient were assumed 600-700 microstrain and 1.0, respectively, for calculating losses. Delplace et al (2008) designed precast thin shells made of UHPFRC for a large roof in a wastewater treatment plant near Paris, France. The total shrinkage strain and creep coefficient were assumed 700 microstrain and 1.0, respectively, for calculating losses.

Based on the results of a research project, Flietstra (2011) developed a prediction model for the UHPC shrinkage strain after the 28-day age. Eq. 5.3.2-3 was proposed as a way to predict the shrinkage strain of UHPC under ambient curing, where “t” is the time after demolding, in days. Mullen (2013) analyzed the experimental results conducted by Flietstra (2011) and developed a best fit equation for creep prediction of UHPC. Eq. 5.3.2-4 was proposed to predict the creep strain of UHPC under ambient curing, where “t” is the time after loading, in days. Mullen (2013) used these prediction models to predict the prestress losses. He applied those models to calculate the losses of three different girders that were tested by former researchers. These models predict the creep/shrinkage strain at any age of UHPC under ambient curing. Based on Flietstra’s research results, Mullen assumed that the residual creep and shrinkage losses will take place within the process of thermal treatment, which is applied after releasing the strand force. Any shrinkage or creep losses that occur before applying the thermal treatment, could be estimated by predicting the creep and shrinkage strain models. He proposed that shrinkage strain will not be increased during the process of thermal curing, which does not match Filetstra’s results. The time dependent losses due to creep and shrinkage were computed using Eq. 5.3.2-5.

\[ \varepsilon_{sh} = 58.7 \ln(t) + 122 \text{ (microstrain)} \]  
\[ \varepsilon_{cr} = \frac{t^{0.6}}{4.069+t^{0.6}} \times 1713 \text{ (microstrain)} \]  
\[ \Delta P_{CR&SH} = \frac{E_{ps} \varepsilon_{cr} + \varepsilon_{sh}}{1000.000} \text{ (ksi)} \]

AFGC (2013) accounts for the UHPC creep effect on long term deformation by dividing the modulus of elasticity by (1+\( \psi \)), which is in agreement with AASHTO LRFD 8th C5.12.5.3.6-1. Mullen (2013) reported that an incremental time step approach could be applied to UHPC. He also reported that there is no noticeable growth in deflection after thermal curing.

### 5.4 Proposed Design Procedure

#### 5.4.1 Prestress Loss

The preceding section shows that the curing method affects shrinkage and creep, which in turn affects the prediction of prestress losses and camber growth. Two curing methods are proposed for structural design. The first method is to cure the concrete prior to prestress release according to PCI MNL 116 requirements, either by moisture retention without heat, or by accelerated heat curing using live steam or radiant heat and moisture, but with internal temperature not exceeding 180°F (82°C). The second method is the same as the first method, in addition to applying thermal treatment in storage, after releasing the prestress. Thermal treatment shall comply with the **UHPC Materials Guide Specification** that was prepared as part of this project.
The total shrinkage can be divided into two components for structural design:

**Initial shrinkage, $\varepsilon_{shi}$**: Shrinkage that takes place between the time of concrete placement and the end of curing. This value is used to calculate initial prestress loss, and the concrete stresses and camber at time of prestress release. It includes most of the autogenous shrinkage.

**Long term shrinkage, $\varepsilon_{shLT}$**: Shrinkage that takes place between the end of curing and time infinity. This value is used to calculate effective prestress to be used for stresses due to full-service loads and for camber growth after prestress release.

**Total shrinkage, $\varepsilon_{us\infty}$**: The sum of the two components defined above.

*Table 5.4.1-1* shows the proposed initial and long-term shrinkage strains, and the proposed creep coefficient at final time, assumed = 20,000 days. These values are tentative, and will be refined in Phase II, based on results of the shrinkage test within this project, as well as the investigation of creep and shrinkage effects, performed by Ohio State University (OSU). The Ohio State University research project is an affiliated project, also sponsored by PCI.

**Table 5.4.1-1. Shrinkage Strain and Creep Coefficient for Structural Design**

<table>
<thead>
<tr>
<th></th>
<th>PCI MNL 116 Curing</th>
<th>PCI MNL 116 Curing Plus Thermal UHPC Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_{shi}$</td>
<td>600 microstrain</td>
<td>600 microstrain</td>
</tr>
<tr>
<td>$\varepsilon_{shLT}$</td>
<td>300 microstrain</td>
<td>0 microstrain</td>
</tr>
<tr>
<td>$\varepsilon_{shu}$</td>
<td>900 microstrain</td>
<td>600 microstrain</td>
</tr>
<tr>
<td>Creep Coefficient</td>
<td>1.2</td>
<td>0.3</td>
</tr>
<tr>
<td>$\psi_u$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Initial prestress losses consist of the component ($\Delta f_{PES}$) due to elastic shortening, and a second component ($\Delta f_{pshi}$) is due to initial autogenous shrinkage. The early age shrinkage effect is represented by the same formulation as AASHTO-LRFD’s current formula for effect of shrinkage in the precast-only component of the cross section, except that the transformed section factor $K_d$ described below, does not have a long-term modifier.

$$\Delta f_{PES} = \frac{E_p}{E_{ct}} f_{cgp}$$  \hspace{1cm} Eq. 5.4.1-1

where

- $f_{cgp}$ = concrete stress at the center of gravity of the prestressing tendons, due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)
- $E_p$ = modulus of elasticity of prestressing steel (ksi)
- $E_{ct}$ = modulus of elasticity of concrete at transfer or time of load application (ksi)

$$\Delta f_{pshi} = \varepsilon_{shi} E_p K_d$$  \hspace{1cm} Eq. 5.4.1-2

where

- $K_d$ = transformed section coefficient that accounts for initial (elastic) interaction between concrete and bonded steel in the section being considered
Thus, the initial prestress to be used in calculating initial concrete stresses and camber is:

\[ P_0 = P_i - A_p(\Delta f_{ES} + \Delta f_{psh}) \]

It is important to note that conservative (high) estimates of concrete stresses due to initial loading (prestress plus member weight) can be obtained by ignoring the second term due to autogenous shrinkage. At final time, the opposite is true. It is conservative at final time (at 20,000 days) to estimate prestress losses on the high side so that the effective prestress is estimated conservatively low and it does not balance as much of the applied gravity loads.

Thus, it may be reasonable to perform analysis according to the current AASHTO procedure with early shrinkage ignored for initial stresses and total ultimate shrinkage fully considered for final stress analysis (Service Levels I and III).

The only modification to the current AASHTO procedure is determination of the coefficients relevant to UHPC materials. The modifications are subject to Phase II studies. They are the factors for the effect of concrete strength, \( k_f \), the humidity factor for creep, \( k_{hc} \), the humidity factor for shrinkage, \( k_{hs} \), and the factor for the effect of the volume-to-surface ratio of the component, \( k_v \). The values of \( k_f, k_{hc}, k_{hs} \) and \( k_v \) are expected to be equal to 1.0 for UHPC.

The example in Appendix F shows the details of calculation of prestress loss. It is recommended to use transformed section properties in all stress analysis. Accordingly, there is no need for explicit calculation of elastic shortening loss. If one applies the prestress force just before transfer and the member weight moment at any given section to the transformed section properties of that section, the correct concrete stresses result. Similarly, any elastic “gain” due to application of deck weight, superimposed dead load and live load, do not have to be computed separately. They can be automatically accounted for when the transformed section properties are used. Only the long-term prestress loss due to creep, shrinkage and relaxation should be counted and applied to the section as “negative prestress.” That force may be added to the initial prestress, just before de-tensioning, to obtain the effective prestress at final time, assumed = 20,000 days.

### 5.4.2 Camber Growth

Initial camber at time of prestress release should take into account the effect of initial autogenous shrinkage. The same applies to initial member shortening. To account for long-term effects, the initial (elastic) camber due to prestress should be multiplied by a factor of \((1 + 0.7\psi)\) to account for losses. The initial deflection due to member weight and other dead loads should be multiplied by \((1 + \psi)\). The symbol \(\psi\) is the creep coefficient for the specific time at loading and for the loading duration. The results from the Ohio State University research project will provide more information to estimate camber growth over time.

### 5.5 References


Time Dependent Effect


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6 BOND BEHAVIOR

6.1 Notation

- $d_b$: nominal diameter of the prestressing strand (in.)
- $f_c$: compressive strength of concrete for use in design (ksi)
- $f_{ci}$: compressive strength at time of release (ksi)
- $f_{pe}$: effective prestressing after losses (ksi)
- $f_i$: initial prestressing stress in the strand immediately after a release (ksi)
- $f_m$: stress in prestressing steel at nominal flexural strength (ksi)
- $f_{pu}$: specified tensile strength (ksi)
- $f_y$: yield strength (ksi)
- $k$: coefficient for AASHTO LRFD 4th Ed. equation, equal to 1.0 for pretensioned panels, piling, and other pretensioned members with a depth of less than or equal to 24.0 in.
- $L_e$: embedment length (in.)
- $l_d$: minimum development length (in.)
- $l_s$: lap splice length (in.)
- $P_u$: ultimate load at failure (kips)

6.2 Introduction

While the compression force due to the bending moment is resisted by the concrete in flexure for continuously-reinforced concrete members, the tension force is mainly resisted by the reinforcement. The available bond stress between concrete and reinforcement provides the required composite action mechanism. Most codes and international practices specify the minimum development length, $l_d$, to reasonably assure that the reinforcement will yield before being pulled out from the concrete. Therefore, the force in a bar, divided by the product of the perimeter and the development length of the bar, should not exceed the concrete bond strength. UHPC provides high bond strength due to its constituent materials and dense matrix. It is also expected to have a shorter transfer length for pre-stressing strands, which is the length from the end of the member, where the tendon stress is zero, to the point along the tendon where the prestress is fully effective (ACI CT-13).

6.3 Previous work

Yuan and Graybeal (2014) have investigated the bond behavior of deformed reinforcing steel in UHPC. The main goal of this study was to provide guidelines when anchoring or lap splicing deformed reinforcing bars in cast-in-place connections. This study investigated the effect of the bar type and size, embedment length, side cover, bar spacing, and UHPC compressive strength, utilizing the pull-out test. Two grades of reinforcing steel were used (i.e., ASTM A1035 Gr. 120 and ASTM A615 Gr. 60) with the bar size ranging from No. 4 to No. 8. A majority of the tests were conducted on specimens with ASTM A1035 reinforcement, to avoid rebar yielding. ASTM A615 reinforcing bars were black steel with an epoxy coating. The embedment length and side cover varied as a function of the rebar diameter, $d_b$. All mixtures had a fiber content of 2%, by volume. The average UHPC compressive strengths were 13.5, 19.4, and 21.3 ksi at age of one, seven, and fourteen days, respectively. Most of the specimens were tested after one or seven days after casting. The effect of casting orientation was also investigated.

The average bond strength was calculated by dividing the bond force at time of failure by the overall contact area. The epoxy rebar showed a lower bond strength than the uncoated rebars. The bond strength decreased by increasing the bar diameter. It was determined that the higher the compressive strength, the higher the bond strength. However, the correlation between the bond and compressive strengths showed a correlation coefficient as low as 0.55 to 0.75. It was also reported that the non-contact lap splice provides a higher bond strength, as long as the clear spacing between two adjacent bars is less than the projection of the crack on the plane perpendicular to the bar length. There was not a noticeable effect from the cast orientation for five out of the six specimens. The test results also showed a lower average bond strength for the specimens that were close to the casting point, than specimens at other locations.

General design recommendations were reported for deformed rebar achieving the lesser of either the yield strength at bond failure or 75 ksi. It was proposed to provide at least $8d_b$ of embedment length in UHPC, with a minimum compressive strength of 13.5 ksi for bar sizes of No. 4 to No. 8. There should also be a minimum side cover of $3d_b$ and a bar clear spacing between $2d_b$ and lap splice length, $l_s$. For UHPC, the side cover can be reduced to $2d_b$ so long...
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as the embedment length is increased to 10d_b. The lap splice is proposed to be a minimum of 75 percent of the embedment length. It was also proposed to reduce the embedment length for applications with a higher compressive strength and/or a larger side cover.

In addition to the recommendation by Yuan and Graybeal (2014), FHWA-HRT-14-084 specified a clear spacing between adjacent discrete reinforcements of at least 1.5 times the length of the longest fiber reinforcement in the UHPC, to allow for appropriate flow. This publication provides a design and construction guidance for field-cast UHPC connections. It also showed some details for connections developed by different DOTs. These connections had a lap splice length ranging from 4.0 to 6.0 in.

Hegger and Bertram (2008) investigated the shear capacity of the UHPC prestressed bulb tee. This study was sponsored by the German Research Foundation (DFG) to develop design guidance for prestressed beams. Hegger and Bertram highlighted the importance of studying the minimum required embedment length of the strands that extend beyond the supports. The embedment length is defined as the length of reinforcement, or anchor, that extends beyond a critical section, over which the transfer of force between concrete and reinforcement may occur (AASHTO LRFD). Due to the arch action, the bottom strands act as a tension tie. This strut and tie behavior enhances the shear strength of the beam. This is in agreement with the modified compression field theory adapted by AASHTO LRFD. The minimum required embedment length is defined as the transfer length (AASHTO LRFD Art. 5.7.3.4.2). Therefore, Hegger and Bertram studied the anchorage behavior of embedded strands in UHPC, by testing 72 pull-out specimens. They studied the effect of the fiber content (i.e. 0, 0.9, 1.4, 2.5% by volume). The specific concrete cover varied in range of 1.5 to 5.5d_b. The prestressing force was released at either three or fourteen days to investigate the effect of the concrete strength. They also investigated two strand sizes of 0.5-in. and 0.6-in. Three prestressing levels were studied to evaluate the wedge action due to the radial expansion of the strands near the free ends at release, which is known as the Hoyer effect. Most of the specimens were tested at three days from casting, with an average compressive strength of 14.5 ksi.

Hegger and Bertram reported that the fiber content has an insignificant effect on the bond strength of UHPC. The average bond strength of fully prestressed specimens with a cover of 4.4d_b was 4.35 ksi, at an age of three days. The strength was reduced by 33% and 60% for specimens with 50% partial prestressing and no prestressing, respectively. For the specimens with a concrete cover less than 2.5d_b the bond strength was significantly reduced, and had a visible splitting crack. The average bond strength of fully prestressed specimens with a cover of 1.5d_b was 2.90 ksi at an age of three days. The effect that reducing the concrete cover had on bond strength was reduced by decreasing the level of prestressing. No noticeable difference in bond behavior was reported between 0.5-in. and 0.6-in. strands. The increase of the compressive strength increased the bond between the strands and the UHPC.

The transfer length was also investigated by casting 10 prisms with 0.5-in. strands that were placed at different spacings and covers. The prestressing force was released at five equal steps to monitor any split cracking. The splitting crack appeared at prestressing levels of 70% and 95% for specimens with a clear cover of 1.5d_b and 2.0d_b respectively. The concrete strain was monitored over the length of the specimens. The transfer length was determined at the point from the specimen end, after which there was no increase in concrete strain. The average transfer length was 20d_b and 32 to 40d_b for uncracked and cracked specimens, respectively.

John et al. (2011) tested seven UHPC beams with dimensions of 6.5-ft by 12-ft by 18-ft, reinforced with 0.6-in. low-relaxation prestressing strands. This study investigated the transfer and development length of prestressing strands in UHPC. They measured the transfer length from the end of the beam to the point with 95% of the average maximum concrete strain. The measured transfer lengths were compared with the predicted values, in accordance with AASHTO LRFD 4th Edition, NCHRP Report 603, and Gowripalan and Gilbert (2000). All prediction models are a function of the size of the strands. The AASHTO LRFD (2007) model was \( f_{pe} / 3d_b \), where \( f_{pe} \) is the effective prestressing after losses, in ksi, and \( d_b \) is the nominal diameter of prestressing strand, in inches. AASHTO LRFD allowed for a simplified calculation of 60d_b. NCHRP Report 603 proposed a prediction model for high strength concrete to be a function of the compressive strength at the time of release, \( 120 / \sqrt{f_{c'}^u} \), where \( f_{c'}^u \) is the compressive strength at time of release, in ksi. The NCHRP recommendation was based on a compressive strength of 9.0 ksi at release. Accordingly, the recommended transfer length is 40d_b. John et al. (2011) have dropped this condition for UHPC. Gowripalan and Gilbert (2000) proposed a transfer length between 20d_b and 40d_b. The measured transfer length was ranging from 18d_b to 30d_b. The predicted values, using the AASHTO LRFD models, show larger transfer lengths than the other models by NCHRP and Gowripalan and Gilbert.

John et al. (2011) have also tested the beams in flexure. The beams were simply supported and a point load was applied near one end of the beam. The shorter distance between the end of the beam and the point of loading is
considered to be the embedment length. The mode of failure showed if the embedment length was adequate for developing the strands or not. If the strands slipped before the beam reached its flexural strength, it was determined that the embedment length is less than the development length. All beams with an embedment length equal to or larger than 42 in. reached their flexural strength before slip occurred. The embedment length was compared to the predicted values, in accordance with AASHTO LRFD 4th Edition, Eq. 6.3-1, and NCHRP Report 603, Eq. 6.3-2. It was reported that both prediction models significantly overestimate the development length of the prestressing strands.

\[ l_d = k \left[ f_{ps} - \frac{2}{3} f_{pe} \right] d_b \]  
\[ l_d = \frac{120}{\sqrt{f_{ci}}} - \frac{225}{\sqrt{f_c}} d_b \geq 100d_b \]  

where  
\[ k = 1.0 \] for pretensioned panels, piling, and other pretensioned members with a depth of less than or equal to 24.0 in.  
\[ f_{ps} = \] stress in prestressing steel at nominal flexural strength (ksi)  
\[ f_{pe} = \] effective stress in prestressing steel after losses (ksi)

Shin et al. (2018) studied the bond behavior of low relaxation pretensioned strands embedded in UHPC. In this study, both the average bond strength and the transfer length were investigated. The average compressive strength of UHPC, utilizing steam curing, was 26.6 ksi. The bond strength was evaluated by pull-out of the strands embedded in 6-in. by 6-in. UHPC cubes, as shown in Figure 6.3-1. The variables were strand diameters, \( d_b \), of 0.5-in. and 0.6-in., prestressing levels of 0.8\( f_{pu} \) and 0.9\( f_{pu} \), embedment lengths, \( L_e \), of 1\( d_b \) and 2\( d_b \), and side covers of 1\( d_b \), 2\( d_b \), 4.4\( d_b \), and 5.4\( d_b \). The prestress levels were selected to be equivalent to 0.72\( f_{pu} \) and 0.81\( f_{pu} \) to cover the typical jacking stress of 0.75\( f_{pu} \) and the maximum allowable stress of 0.8\( f_{pu} \). Due to the high expected bond strength of UHPC, RILEM (1994) recommended an embedment length of 5\( d_b \) to avoid splitting failure. The embedment length from Shin et al. (2018) was much lower than that. Unlike Yuan and Graybeal (2014), all specimens failed due to pull-out of the strands without any visible splitting cracks. The average bond strength was obtained by dividing the ultimate load at failure \( (P_u) \) by the surface area \( (\pi d_b L_e) \) of the embedment length. Specimens with 0.5-in. strands obtained a 45% higher bond strength than those with 0.6-in. strands. The average bond strength was reported as 3.6 and 5.2 ksi for 0.6-in. and 0.5-in. strands, respectively. The UHPC bond strength, with 0.6-in. strands, is 2.8 times the bond strength of conventional concrete with strands that was reported by Girgis and Tuan (2005). Based on Yoo et al. (2014), Shin et al. (2018) reported that the average UHPC bond strength with No. 5 deformed bars is 175% higher than with 0.6-in. strands, which is attributed to the interlock between the ribs of the rebar and the concrete. Contrary to Hegger and Bertram (2008) findings, the prestress level had an insignificant effect on the bond strength. This might be attributed to the small difference between the two prestress levels in this study. Increasing the side cover from 1\( d_b \) to 2\( d_b \) increased the average bond strength for 0.6-in. and 0.5-in. strands. Increasing the embedment length from 1\( d_b \) to 2\( d_b \) decreased the average bond strength for 0.6-in. and 0.5-in. strands.
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The variables in the transfer length test were 0.5-in. and 0.6-in. strands with a side cover of 1\(d_b\), 2\(d_b\), or 3\(d_b\). Figure 6.3-2 shows the test setup of the transfer length test. Strain gauges were attached to the strands to monitor strain variations. The transfer length was determined based on a 95% average maximum strain. It was reported that the transfer length at the dead end was shorter than at the live end. The test results showed that the larger the diameter, the longer the transfer length became. The average measured transfer lengths were 19\(d_b\) and 24\(d_b\) for 0.5-in. and 0.6-in. strands, respectively. Increasing the side cover also increased the transfer length. Shin et al. (2018) compared the measured transfer length with seven prediction models: ACI 318-14, AASHTO LRFD 6th Ed., Russell and Burns (1996), Mitchell et al. (1993), Eurocode 2 (2004), Martí-Vargas et al. (2007), and Dang et al. (2016). While ACI and AASHTO LRFD significantly overestimated the transfer length, the model from Mitchell et al. (1993) provided an adequate prediction. This model uses the equation: \(\frac{1}{3} f_{pi} d_b \sqrt{2.9 f_{ci}' / f_{ci}}\), where \(f_{pi}\) is the initial prestressing stress in the strand immediately after a release, in ksi, and \(f_{ci}'\) is the initial compressive strength of concrete at the time of release, in ksi. The transfer length was predicted as 23\(d_b\) for both 0.5-in. and 0.6-in. strands using the model provided by Mitchell et al. (1993).

Graybeal (2014) investigated the development length of untensioned 0.5-in. and 0.6-in. prestressing strands embedded in UHPC reinforced with 2%, by volume, of steel or polyvinyl alcohol (PVA) fibers. The intent of this research was to study the behavior of non-contact lap splices of untensioned prestressing strands for field-cast UHPC connections. The UHPC specimens had a compressive strength of 23.2 and 19.6 ksi for specimens with steel and PVA fibers, respectively. The tensile cracking strength ranged from 1.16 to 1.45 ksi. Figure 6.3-3 shows concrete dimensions and the test setup of the development length test. It was reported that the clear spacing between the strands is smaller than the 1.5 times the fiber length specified by AFGC (2013). AFGC recommended this limit to provide an adequate flow. The concrete cover and spacing were controlled by the common practice in
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**Bond Behavior**

US prestressing construction, such that the strands are placed on a 2-in. by 2-in. grid. The reference group of specimens were made of UHPC with steel fibers, with 0.5-in. strands with embedment lengths (splice lengths) of 8, 12, 16, 20, and 24 inches. A similar group was cast with PVA fibers, and another group with 0.6-in. strands. An additional group was cast for UHPC with PVA, with an embedment length of 24, 30, and 36 inches. The strand was considered to be fully developed when the measured tensile stress of the strand reached the specified minimum strand strength of 270 ksi. Most of the specimens failed due to splitting cracks. The test results showed that untensioned 0.5-in. and 0.6-in. prestressing strands can be fully developed in UHPC containing steel fibers, with an embedment length of 40\(d_b\). Graybeal also reported a development length of approximately 72\(d_b\) for 0.5-in. strands embedded in UHPC with PVA.

![Test Setup of Development Length Test](graybeal_2014.png)

**Figure 6.3-3. Test Setup of Development Length Test (Graybeal, 2014) (Note: 1 mm = 0.0394 in.)**

**6.4 Pilot Testing: Development Length Components**

Strand development specimens were provided by three different manufacturers, Precasters A, B, and D, as previously designated for this project. The test specimens were fabricated in each of manufacturers’ facilities using the individual UHPC mixes developed. The specimen geometry was similar to that used by Graybeal (2014). Details of the specimens are shown in **Figure 6.4-1**. Four specimens were supplied by each precaster with un-tensioned strands and four with strands pretensioned to 75% of ultimate strength of 270 ksi. The four specimens have lengths, \(L\), of 6, 12, 18, and 24 in. The strand spacing, \(X\), depended on the size used in their product. The strand spacing were 1.5” and 2” for 9/16” and 0.6” strands, respectively.

![Specimens Used in Development Length Testing](specimens.png)

**Figure 6.4-1. Specimens Used in Development Length Testing**
Precaster A provided only unstressed specimens. Precasters B and D provided both unstressed and prestressed strand specimens. Concrete compressive strength from Precaster A was reported as 24.19 ksi at 28 days and from precasters B and D as 18.4 and 17.7 ksi, respectively. Concrete peak flexural tensile strength using ASTM C1609 standard testing averaged 3.1 ksi at 28 days.

A load rate of approximately 18,000 pounds per minute was utilized. For each test, the load, displacement (slip) from each LVDT, and crosshead travel were recorded. During testing, a plot of load versus average slip and a plot of load versus crosshead movement were observed. Once the LVDTs were no longer in contact with the concrete surface (after approximately 0.2 inches of slippage), they were removed to avoid damage at strand failure. Peak strand stress was calculated using the ultimate load and dividing by the nominal area for the determined strand diameter. Failure load, corresponding strand stress, and failure mode are reported in Table 6.4-1. Observed failure modes included 1) slip of the strand from the specimen, 2) slip of the strand from the specimen and concrete splitting crack along the center of the specimen, 3) strand failure greater than 1/2 inches from chuck, and 4) strand failure at the chuck.

<table>
<thead>
<tr>
<th>Fabricator</th>
<th>Strand Diameter (in.)</th>
<th>Strand Embedment Length (in.)</th>
<th>Strand Embedment/strand diameter</th>
<th>Prestress Failure Load (kip)</th>
<th>Failure stress (ksi)</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precaster B</td>
<td>9/16</td>
<td>6</td>
<td>10.7</td>
<td>0</td>
<td>15.5</td>
<td>80.7</td>
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<tr>
<td>Precaster B</td>
<td>9/16</td>
<td>12</td>
<td>21.3</td>
<td>0</td>
<td>37</td>
<td>193.2</td>
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<tr>
<td>Precaster B</td>
<td>9/16</td>
<td>18</td>
<td>32.0</td>
<td>0</td>
<td>46</td>
<td>237.0</td>
</tr>
<tr>
<td>Precaster B</td>
<td>9/16</td>
<td>24</td>
<td>42.7</td>
<td>0</td>
<td>50</td>
<td>262.0</td>
</tr>
<tr>
<td>Precaster B</td>
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<td>10.7</td>
<td>38.9</td>
<td>18</td>
<td>94.8</td>
</tr>
<tr>
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<td>21.3</td>
<td>38.9</td>
<td>29</td>
<td>148.4</td>
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<tr>
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<td>18</td>
<td>32.0</td>
<td>38.9</td>
<td>51</td>
<td>267.2</td>
</tr>
<tr>
<td>Precaster B</td>
<td>9/16</td>
<td>24</td>
<td>42.7</td>
<td>38.9</td>
<td>52</td>
<td>268.2</td>
</tr>
<tr>
<td>Precaster D</td>
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<td>10.7</td>
<td>0</td>
<td>15</td>
<td>79.2</td>
</tr>
<tr>
<td>Precaster D</td>
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<td>12</td>
<td>21.3</td>
<td>0</td>
<td>35</td>
<td>183.9</td>
</tr>
<tr>
<td>Precaster D</td>
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<td>32.0</td>
<td>0</td>
<td>50</td>
<td>259.9</td>
</tr>
<tr>
<td>Precaster D</td>
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<td>24</td>
<td>42.7</td>
<td>0</td>
<td>52</td>
<td>270.8</td>
</tr>
<tr>
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<td>6</td>
<td>10.0</td>
<td>0</td>
<td>20</td>
<td>91.7</td>
</tr>
<tr>
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<td>0</td>
<td>36</td>
<td>165.0</td>
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<td>0</td>
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<td>40.0</td>
<td>43.9</td>
<td>57</td>
<td>263.1</td>
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</tbody>
</table>

Figure 6.4-2 shows the strand stress versus embedment length/strand diameter ratio. As expected, the capacity of strand to take tension increases with increased embedment length. The results from the various precasters follow a narrow band despite the difference in mixture proportions and concrete strength. It is believed that the most important factor in all mixes is that they all have the same high strength, high aspect ratio fibers in the same quantity as 2 percent by volume. Another observation is that prestress did not seem to have a significant impact on improving the ability of the strands to further bond to the concrete due to the Hoyer effect. To further illustrate the impact of prestress, see Figure 6.4-3. It shows that there is improvement in bond but not significant (less than 4 percent).
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Figure 6.4-2. Failure Load versus Embedment Length, Expressed as Number of Strand Diameters

Figure 6.4-3. Results Showing Minor Improvement in Bond Strength between Strand and Concrete Caused by Prestress

6.5 Proposed Tentative Design Recommendations

The transfer length of prestressing strands effects the vertical shear behavior, the stress distribution, and the bursting forces of any prestressed member. AASHTO LRFD 7th Ed. extended the 60d_b transfer length to be applied to normal weight conventional concrete with specified compressive strengths up to 10.0 and 15.0 ksi at release and final, respectively, based on NCHRP Report 603. These compressive strengths are close to what is specified in this project. However, the constituent materials of UHPC are different than that of conventional concrete, CC. This results in a higher bond strength for UHPC than CC. It is proposed to assume that the transfer length ranges from 20d_b to 40d_b (to be confirmed in Phase II) with a minimum clear cover and spacing of 2d_b.

Conservatively, the transfer length should be assumed as the lower and upper bound for anchorage zone and vertical shear calculations, respectively. The shorter the transfer length, the higher the stress concentration at release. This introduces higher transverse stresses at the anchorage zone, thus increasing the demand of the resisting reinforcement. For shear design, a shorter transfer length results in a higher shear capacity at the same section that is near the end of the beam. If the section under consideration is being checked for shear strength and is located within the transfer length, the prestressing force should be reduced, assuming that the stresses are
transferred linearly over the transfer length, and increase from zero, at the location where the bond between the strands and concrete commences, to its full value at the end of the transfer length (AASHTO LRFD). The lower bound may also be conservatively used to check against the stress of the element top fibers at time of release.

When all pilot test results are combined and approximated into a design approach similar to that of the development length equation in AASHTO for conventional concrete, the result is a bilinear relationship as shown in Figure 6.5-1. To reach a stress of 170 ksi which is approximately equal to the effective prestress in prestressed UHPC members, a strand embedment of 20 strand diameters is needed. To reach the ultimate strength of 270 ksi, an embedment length of 40 strand diameters is needed.

A format, similar to that in AASHTO would be:

\[ L_d = L_t + 0.2d(f_{ps} - f_{pe}) \]

where

\[ L_t = 20d_b \]

Eq. 6.5-1 would need to be further developed and verified with a larger sampling of test results than shown here. Some additional tests are expected to be available in Phase II of this project.

![Figure 6.5-1. Bilinear Strand Development Length in Terms of Number of Strand Diameters](image)

In Phase II, consideration will also be given to isolating specimen web width as a test parameter for members whose webs include strands. The purpose would be to determine the web width at which horizontal splitting may limit development.

As can be seen in Figure 6.4-3, the test results show minor improvement in bond strength for prestressed strand compared to non-prestressed strand. The difference may be due to the Hoyer effect, the radial expansion of strand following release, which increases bond due to “wedge action.” On the other hand, the minor difference could be due to the inherent variability in specimen fabrication and testing. Regardless, the difference does not justify separately testing prestressed strand in Phase II tests.

### 6.6 Plans for Phase II Testing

#### 6.6.1 Decked I-Beam Longitudinal Connection

Some structural details are related to the bond behavior of UHPC with embedded reinforcing. This includes the concrete cover, the spacing of continuous reinforcement, the embedment length, and the splice length. The recommendations of these details are limited to the longitudinal connection between two adjacent bridge girders. Figure 6.6.1-1 shows the proposed details of the transverse reinforcement in the deck of the decked I-beam, within the closure joints. The proposed details for the transverse reinforcement of the box slab is similar. These details are
not in line with the recommendations by Yuan and Graybeal (2014). The proposed details will be tested and reported in Phase II.

Figure 6.6.1-1. Transverse Non-Contact Splice for Decked I-Beam Top Flange

Two new ribbed slab pieces of the decked I-beam will be produced. The two pieces will be transversely connected by a second casting of UHPC, to create a longitudinal joint. The joint capacity will be tested as shown in Figure 6.6.2-1. This test will investigate the flexural strength of the ribbed slab when the top flange is subjected to normal compression stresses. It should be noted that this test may be eliminated if the research team is able to have a timely report of the results of the same type of test. This test is planned to be performed by the University of Windsor for FACCA, as part of the verification of the design by e.Construct for the Hitch House Bridge to be built near Toronto, Ontario.

Figure 6.6.2-1. Alternative Longitudinal Joint Test Setup

6.6.2 Decked I-Beam Longitudinal Connection

The objective of this testing is to evaluate the capacity and structural performance of UHPC longitudinal and transverse connections between UHPC bridge components. UHPC box slabs will be used in this investigation due to their constructability and structural efficiency for short and medium span bridges. Figure 6.6.2-1 shows the proposed UHPC box section. The product will be made by Standard Concrete Products, as the planned shape is the
same shape being considered for possible implementation in Florida. The longitudinal connection between adjacent box beams will be tested. Figure 6.6.2-1 shows the setup for testing each connection. Two 10-ft long UHPC box beams will be produced by SCP, and shipped to the UNL structural laboratory for testing. It should be noted that the exterior web surfaces of adjacent beams will be prepared to provide adequate adhesion between the UHPC precast segments and the cast in place UHPC. Testing will be performed to establish the joint performance when corrosion resistant rebars are used in the joint, in place of Grade 60 black steel. The testing will determine the joint capacity with these bars, and will evaluate the ability of these bars to be fully developed in the relatively narrow joint being proposed.

![Figure 6.6.2-1. UHPC Box Beam Test Setup for the Longitudinal Joint](image)

6.6.3 Transfer Length Measurement
The transfer length will be measured at the time of strand release for all test specimens. The specimens will include bridge and building beams with 9/16 and 0.6-in. strands. It is envisioned to mount DEMEC mechanical strain gauges on both sides and both ends of any beam, close to the center of gravity of the strands, before strand release, as shown in Figure 6.6.3-1. The DEMEC points will be mounted over a total length of 80 times the strands diameter. The strain will be measured at the time of transfer and at ages of 7, 14, and 28 days. By plotting strain versus distance form each end, the transfer length will be determined as the distance from the end of the beam, to the point where strain no longer changes.
6.6.4 Development Length Components

It was envisioned to evaluate strand development using tension tests similar to Graybeal (2014), as shown in Figure 6.6.4-1. In addition to the pilot testing, more specimens will be fabricated in each of manufacturers’ facilities using the individual UHPC mixes developed. Each participating precaster will cast two groups of test specimens and send them to WJE for testing. One group will have four specimens with untensioned strands and embedment lengths of 6, 12, 18, and 24 in. The other group is typical, but with tensioned strands. The specimen will have both a center-to-center spacing and a side cover of 1.5 in., for 9/16-in. strands. For 0.5-in., 0.6-in., and 0.7-in. strands, the strands will be placed on a 2.0-in. grid. The embedment length will be considered as the development length in the case of reaching the strand’s minimum specified ultimate tensile strength of 270 ksi, before a slip of 0.01 in.
6.7 References

ACI Committee 318. (2014). Building Code Requirements for Structural Concrete and Commentary, (ACI 318R-14), American Concrete Institute: Farmington Hills, MI.


7 VERTICAL SHEAR DESIGN

7.1 Notation

- $A_{ps}$ = area of prestressing steel (in.$^2$)
- $A_s$ = area of nonprestressed tension reinforcement (in.$^2$)
- $a$ = shear span (in.)
- $a_g$ = aggregate size (in.)
- $b_v$ = effective web width taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the tensile and compressive forces due to flexure (in.)
- $b_w$ = width of member’s web (in.)
- $d$ = effective depth of the member defined as the distance between the extreme compression fiber and the centroid of the primary longitudinal reinforcement (in.)
- $d_n$ = nominal strand diameter (in.); nominal diameter of reinforcing bar (in.)
- $d_e$ = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.)
- $d_f$ = diameter of fiber (in.)
- $d_d$ = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement measured along the centerline of the web (in.)
- $d_v$ = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure (in.)
- $E_{ps}$ = modulus of elasticity of prestressing steel (ksi)
- $E_s$ = modulus of elasticity of steel reinforcement (ksi)
- $f'_c$ = compressive strength of concrete for use in design (ksi)
- $f_{fu}$ = peak flexural strength in ASTM C1609 Standard Flexural Test (ksi)
- $f_{po}$ = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi)
- $f_{pc}$ = the compressive stress of the concrete at the centroid of the cross-section resisting external loads or at the junction of web and flange when the centroid lies within the flange (ksi)
- $f_{ps}$ = average stress in prestressing steel at the time for which the nominal resistance of member is required (ksi)
- $f_{rr}$ = extreme fiber tensile strength in ASTM C1609 Standard Flexural Test occurring in a strain range between first cracking and a crack width of 0.3 mm (0.012 in.) (ksi); post cracking residual tensile strength (ksi)
- $f_t$ = tensile strength of the concrete using direct tension testing (ksi)
- $f_{fi}$ = maximum tensile strength of fiber reinforced composite (ksi)
- $f_{te}$ = effective tensile strength of fiber reinforced composite (ksi)
- $f_y$ = specified minimum yield strength of reinforcement (ksi)
- $h$ = overall depth of a member (in.)
- $K$ = fiber orientation factor
- $K_{t, max}$ = maximum value of the global orientation factor
- $L$ = span length (ft)
- $\ell_d$ = development length (in.)
- $l_f$ = length of fiber (in.)
- $M_o$ = factored moment at the section (kip-in.)
- $N_o$ = factored axial force, taken as positive if tensile and negative if compressive (kip)
- $P$ = axial force (kip)
- $P_e$ = effective force in prestressing steel after losses (ksi)
- $P_n$ = nominal axial resistance (kip)
- $s_x$ = crack spacing parameter (in.)
- $s_{xe}$ = crack spacing parameter as influenced by aggregate size (in.)
- $V_c$ = nominal shear resistance of the concrete (kip)
- $V_{cw}$ = nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web (kip)
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\( V_{\text{exp}} \) = experimental ultimate shear force (kip)
\( V_{\text{cap}} \) = experimental ultimate shear stress (ksi)
\( V_f \) = shear strength fiber factor
\( V_n \) = nominal shear resistance (kip)
\( V_{n_{\text{DTS}}} \) = predicted shear force by the Direct Tension Approach (kip)
\( V_{n_{\text{SMCF}}} \) = predicted shear force by SMCF (kip)
\( V_p \) = component of prestressing force in the direction of the shear force (kip) (5.7.2.3)
\( V_s \) = shear resistance provided by transverse reinforcement
\( \nu \) = tensile effectiveness factor
\( w \) = crack mouth opening displacement (in.); crack width (in.)
\( x_d \) = crack length (in.)
\( \alpha_f \) = aspect ratio of the fiber
\( \beta \) = factor indicating the ability of diagonally cracked concrete to transmit tension and shear
\( \gamma_f \) = fiber efficiency
\( \delta \) = midspan deflection (in.)
\( \varepsilon_n \) = net longitudinal tensile strain in the section at the centroid of the tension reinforcement (in./in.)
\( \theta \) = angle of inclination of diagonal compressive stresses (degree)
\( \theta_{\text{exp}} \) = experimental angle of inclination of diagonal compressive stresses (degree)
\( \rho_f \) = volumetric fraction of fibers
\( \tau_b \) = bond stress between the fibers and the concrete matrix (ksi)
\( \Phi \) = resistance factor
\( \psi \) = reduction factor for fiber orientation and inverse analysis

7.2 Introduction

Perhaps, the most significant design criteria for design of UHPC flexural members, such as beams and slabs, are those related to vertical shear. There is a combination of high compressive strength due to particle packing, high tensile strength due to the use of 2 percent by volume of high strength fibers, and high ductility due to the ductility of the fibers and their high aspect ratio. This combination of factors leads to high shear strength and excellent service load performance. As seen in separate sections of this report, one can satisfy shear requirements at the critical section of a 250 ft long bridge beam while using only 4-inch wide web and no shear reinforcing bars. Similarly, only 2-inch webs may be needed for a box beam spanning 60 feet. This presents a significant opportunity to optimize precast prestressed concrete products. Reduced labor and quality control for the placement of bars and maintenance of consistent concrete cover to the rebar are parameters that can create savings, in addition to the rebar material savings, which is not the primary source of savings. Also, when no steel shear reinforcement is used, only one section, the critical section near the support, needs to be designed in the entire beam if a constant web width is used as is the case in most precast pretensioned members. In fact, inclusion of vertical bars in narrow webs may disrupt the flow of fibers as the concrete is placed in the forms.

While vertical shear is perhaps the most important parameter in this research in terms of structural optimization, it is the least understood when fibers are used in the concrete mix. Numerous studies, see for example Lantsoght (2019), and several codes, such as ACI 318-14, have already included provisions for utilization of concrete reinforced with steel fibers.

Numerous studies have been conducted in the past 25 years on materials with properties similar to the material being developed for use in this project. These studies and available international codes and guidelines are used in the following sections to help arrive at a tentative recommendation, to be verified with a comprehensive testing program in Phase II. Most of the recommendations are eliminating any provision for minimum shear reinforcement as the concrete and fibers matrix are being counted on. Eliminating such reinforcement provides a clear path in the web without obstructions disturbing the fiber alignment and clogging of the fibers. This, in turn, enhances the efficiency of the fiber contribution to the shear strength.

This section provides overview and comparison between various available models of shear design. It is followed by the tentative proposal at this stage of the project. Examples, and recommendations which are in line with other successful practices.
7.3 Previous Research

Hegger et al. (2004) conducted an experimental program to evaluate strand anchorage and shear behavior of UHPC pretensioned I-beams with compressive strength of 200 MPa (29.00 ksi) without steam curing and 360 MPa (52.21 ksi) with 250 °C (480 °F) steam curing, and flexural tensile strength of 40 MPa (5.80 ksi). Strand bond tests, using 202 MPa (29.30 ksi) concrete revealed considerable improvement with transfer of prestress achieved in about 10 in., or 20 strand diameters, rather than the standard 50 to 60 strand diameters assumed for conventional concrete. The clear concrete cover and strand-to-strand clear spacing can be reduced to about 1.5d to 2.0d. This corresponds to a clear cover of 0.75 to 1 in. and a center-to-center spacing of 1.25 to 1.5 in. for 0.5-in. diameter strands.

Shear tests were conducted by Hegger et al. on 300 mm (12 in.) deep I-beams, with 70 mm (2.76 in.) wide webs. It was noticed that the centerline of the support bearings was only 6 inches away from the end of the beam. Figure 7.3-1 is a photo of the failed beam. It seems to indicate that the failure was actually a combination of shear and bond failure, although Hegger et al characterizes it as shear failure. The shear stress at failure was 1.9 ksi, which is lower than values seen in other tests as discussed below. The horizontal crack at the flange/web junction is curious and somewhat concerning, even if it is a secondary failure. Our research team has intentionally created an I-beam web/flange transition that is much more gradual than most of the investigations performed to date. We will carefully assess the flange/web transition zone.

Graybeal (2006) evaluated the shear performance of UHPC prestressed I-Girders by testing three AASHTO Type II girders with different shear span-to-depth ratio, a/d. Specimen S24 with a/d of 2.5 expressed a clear shear failure with shear stress at failure, \( V_u/(b_w d) \), where \( d \) is the effective depth in flexure, of 2.6 ksi. Graybeal compared the test results with the prediction model of interim recommendation by AFGC (2002) by placing all partial safety factors equal to 1.0. The residual tensile strength and \( \theta \) were conservatively assumed to be 1.0 ksi and 40°, respectively. However, the calculated strength was only half of the experimental value. He reported that the AFGC model significantly underestimates the shear strength of UHPC prestressed members. As will be shown later in Crane (2010), using a flatter angle, as has been demonstrated in following tests (Baby et al., 2010, Carne, 2010, and Voo et al., 2006 and 2010), would bring the experimental and predicted values closer. Also, a less conservative tensile strength assumption would also bring the experimental and predicted values closer together.

Graybeal’s study brought out an important parameter. In the test he identified as 14S, the strands slipped, indirectly pointing out the importance of the tension tie check performed in the AASHTO shear design provisions, but not in the ACI 318 provisions. While the test showed high shear resistance, even with the strand slippage, maximum possible shear capacity cannot be achieved without providing adequate strand embedment length, or other rebar anchorage means, beyond the critical diagonal crack near the support.

Graybeal (2006) further indicated that the development length \( L_d \) of 0.5-in. diameter strand in UHPC appeared to be less than 37 in. Thus, \( L_d/d_b = 74 \). This is about one-half of the 150\( d_b \) typical assumed with conventional concrete. This benefit would result in general reduction of reinforcement embedment length beyond the critical shear crack.

A comprehensive review of the shear resistance of steel fiber reinforced concrete without conventional stirrups was conducted by Khuntia et al (1999). The concrete strength ranged from 3 to 14 ksi. The specimens had a span...
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Baby et al. (2010) conducted a study as part of a Task Group aiming at providing support for the contents of the AFGC (Association Française de Génie Civil), herein called the “French Recommendation”, on UHPPRRC (Ultra High Performance Fibre-Reinforced Concrete). The research was conducted at the French Public Works Research Institute (LCPC) to provide support of shear design in the French Recommendation. They conducted four-point testing of nine 3 m (10 ft) long I-beams with variable amounts of prestressing, passive longitudinal reinforcement, and shear stirrups. All specimens met the 150 MPa (21.75 ksi) compressive strength: tensile stress of 10 MPa (1.45 ksi). They all contained steel fibers of the type used in this research in amounts ranging from 2 to 2.5 percent by volume. The total beam depth was 380 mm (14.96 in.), effective depth was 305 mm (12.01 in.) and web width was 65 mm (2.56 in.). Four beams were provided with shear reinforcement, with a fiber aspect ratio of 60 and 0.49 \( \times d \) for fiber aspect ratio of 100.

The authors indicated that providing a minimum clear concrete cover of 1.5 times the fiber length and special reinforcement configuration minimized the probability of disturbing the flow of fibers into the forms. The stirrups were 8 mm (0.31 in.) in diameter and spaced at 75 mm (2.95 in.) and were intended to introduce a contribution to the shear capacity of about 20 percent.

The authors concluded that the French Recommendation predictions, with resistance factors set at 1.0, produce much lower predicted capacity. The ratio of experimental to theoretical failure load varied from 1.35 to 1.97. **Table 7.3-1** gives a summary of the results and also shows the nominal shear stress at failure, assuming an effective shear depth of 0.8 of the total depth. The ratio is quite impressive; it is over 3.00 ksi in all cases. Observation of the load deflection charts indicates that providing stirrups in the amounts given in the research added a minor load capacity, but contributed significantly to increased ductility.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Longitudinal Reinforcement</th>
<th>Stirrups</th>
<th>Failure Type</th>
<th>Failure Shear Force, ( V_{exp} ) (kip)</th>
<th>( V_{exp}/(b_w.d_e) ) (ksi)</th>
<th>( V_{exp}/V_n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-A</td>
<td>Prestressed</td>
<td>No</td>
<td>Shear</td>
<td>99</td>
<td>3.23</td>
<td>1.36</td>
</tr>
<tr>
<td>1-A-bis</td>
<td>Prestressed</td>
<td>No</td>
<td>Shear</td>
<td>99</td>
<td>3.22</td>
<td>1.35</td>
</tr>
<tr>
<td>1-B</td>
<td>Prestressed</td>
<td>No</td>
<td>Shear</td>
<td>116</td>
<td>3.77</td>
<td>1.58</td>
</tr>
<tr>
<td>2-A-bis</td>
<td>Prestressed</td>
<td>Yes</td>
<td>Shear</td>
<td>125</td>
<td>4.08</td>
<td>1.41</td>
</tr>
<tr>
<td>2-B</td>
<td>Prestressed</td>
<td>Yes</td>
<td>Shear</td>
<td>143</td>
<td>4.67</td>
<td>1.61</td>
</tr>
<tr>
<td>3-A</td>
<td>Reinforced</td>
<td>No</td>
<td>Shear</td>
<td>104</td>
<td>3.38</td>
<td>1.96</td>
</tr>
<tr>
<td>3-B</td>
<td>Reinforced</td>
<td>No</td>
<td>Shear</td>
<td>102</td>
<td>3.33</td>
<td>1.94</td>
</tr>
<tr>
<td>4-A</td>
<td>Reinforced</td>
<td>Yes</td>
<td>Flexure</td>
<td>122</td>
<td>3.99</td>
<td>1.84</td>
</tr>
<tr>
<td>4-B</td>
<td>Reinforced</td>
<td>Yes</td>
<td>Flexure</td>
<td>117</td>
<td>3.81</td>
<td>1.76</td>
</tr>
</tbody>
</table>

Table 7.3-1. Research Results of Baby et al. (2010)
Crane (2010) conducted full scale testing of six I-beams, Tests 1-1a, 1-2, 2-1, 2-2, 3-1 and 3-2. The beams had 32 in. total depth and 3.9 in. web width as shown in Figure 7.3-2. Test 1-1a was performed on the end of girder 1 that had experienced interface failure during a previous flexural test. The deck was then removed from that end and the beam was tested at a shear span of 6 ft, or a span-to-depth ratio of 2.25. The beam had 2#4 stirrups at 24 in. spacing. The beam failed in flexural compression when the shear force was 452 kip. The resulting shear stress is approximately \( \frac{V}{b_v \times 0.8 \times h} = 4.53 \text{ ksi} \).

Test 1-2 and Test 2-1 were identical except that Test 1-2 had a fluted interface with the deck while Test 2-1 had a smooth interface. Both beams had no shear reinforcement. The shear span was 8 ft, representing a shear span-to-depth ratio of 2.3. The shear capacity of Test 1-2 was 431 kip, corresponding to an approximate shear stress \( \nu_v = \frac{V_{test}}{(b_w \times 0.8h)} = 3.42 \text{ ksi} \). Lack of roughening of the interface in Test 2-1 caused deck-beam interface slippage at a relatively low load. Test 2-1 failed in a similar flexural compression failure as Test 1-1a. It reached a shear capacity of 3.70 ksi.

Test 2-2 contained 2#4 at 24 in. and also a fluted interface. It had considerably more stiffness than Test 2-1. It failed in shear at a shear stress of 3.82 ksi.

Beam 3, on which Tests 3-1 and 3-2 were conducted, had a relatively narrow top flange. The first end (Test 3-1) had a fluted interface and no stirrups, the second end (Test 3-2) had a smooth interface and contained stirrups. Both ends exhibited slippage during previous full beam length testing. In shear testing with a short span, they had reduced stiffness similar to that exhibited in Tests 1-1a and 2-1. Test 3-1 failed in flexural compression after reaching a shear force of 422 kips, or 3.35 ksi, assuming an effective shear depth of 0.8*(32+8.39), which may be questionable but conservative. Test 3-2 also failed in flexural compression at the top of the beam with a corresponding shear force of 444 kip, or 3.52 ksi. This compression failure occurred after slippage at the smooth deck/beam interface. This was interesting as the stirrups appeared to have little impact on preventing the slippage. It seems to lead to a conclusion that composite action between the beam and the deck requires careful consideration in both construction and analysis.

Table 7.3-1 shows theoretical predictions using AASHTO LRFD Modified Compression Field Theory with the test results. In the analysis, Crane used approximate equations for the angle of shear crack, \( \theta \) and the \( \beta \)-factor, required for estimating concrete strength. He also proposed use of the French Recommendation to estimate the contribution of the fibers as shown in Eq. 7.3-1.

\[
V_f = b, d, f, \cot \theta \tag{Eq. 7.3-1}
\]

Where \( V_f \) is the vertical shear capacity contributed by the fibers, \( b_w \) is the web width, \( d_v \) is the effective shear depth, assumed to be 0.8h where h is the total depth, and \( \theta \) is the primary shear crack angle. The parameter \( f_{cr} \) is defined as the extreme fiber tensile strength in a standard flexural test occurring in a strain range between first cracking and a crack width of 0.3 mm (0.012 in.). Based on research by Graybeal (2006) the value of \( f_{cr} \) may be taken as 1 ksi.

Despite the fact that several of the tests indicated flexural compression failure before shear failure, the table shows a close correlation with the AASHTO Simplified Modified Compression Field Theory (SMCF) method. It also confirms consistency between theory and experiments relative to shear crack angle and a general conservatism in the theoretical prediction.
Vertical Shear Design

Table 7.3-2. Calculation of Shear Capacity by Simplified Modified Compression Field Theory, Crane (2010)

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>$\theta$</th>
<th>$V_c$ (kip)</th>
<th>$V_s$ (kip)</th>
<th>$V_f$ (kip)</th>
<th>Predicted Shear Force by SMCF, $V_{n, SMCF}$ (kip)</th>
<th>Failure Shear Force, $V_{exp}$ (kip)</th>
<th>$\theta_{exp}$</th>
<th>$V_{exp}/V_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1 a</td>
<td>26.6</td>
<td>142</td>
<td>48</td>
<td>186</td>
<td>376</td>
<td>452</td>
<td>34</td>
<td>1.20</td>
</tr>
<tr>
<td>1-2</td>
<td>27.4</td>
<td>107</td>
<td>0</td>
<td>180</td>
<td>287</td>
<td>431</td>
<td>26</td>
<td>1.50</td>
</tr>
<tr>
<td>2-1</td>
<td>27.3</td>
<td>111</td>
<td>0</td>
<td>181</td>
<td>293</td>
<td>466</td>
<td>23</td>
<td>1.59</td>
</tr>
<tr>
<td>2-2</td>
<td>27.5</td>
<td>104</td>
<td>46</td>
<td>179</td>
<td>330</td>
<td>480</td>
<td>28</td>
<td>1.46</td>
</tr>
<tr>
<td>3-1</td>
<td>27.4</td>
<td>107</td>
<td>0</td>
<td>180</td>
<td>287</td>
<td>422</td>
<td>25</td>
<td>1.47</td>
</tr>
<tr>
<td>3-2</td>
<td>27.6</td>
<td>101</td>
<td>92</td>
<td>179</td>
<td>372</td>
<td>444</td>
<td>34</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Figure 7.3-2. Cross-Section of Precast Prestressed UHPC Girders 1 and 2 (Crane, 2010)

Crane (2010) evaluated shear prediction using what he called “the direct tension model”, reported in Hawkins et al. (2005) for prestressed concrete beams. Note that there is no current ASTM standard for direct tension testing. The nominal shear strength was attributed to two components; concrete and stirrups contribution. The nominal shear strength provided by concrete, $V_{cw}$ includes the vertical components of effective prestress force at section, $V_p$, as shown in Eq. 7.3-2. The first term represents that shear cracking results from high principle tensile stress in the web. The concrete tensile strength, $f_t$, was assumed = 1.40 ksi per Garas (2009) direct tension test results. The nominal shear strength provided by stirrups, Eq. 7.3-3, accounts for the angle of inclination of diagonal compressive strut, $\theta$, Eq. 7.3-4. Crane (2010) presents a comparison between the two prediction models comparing them with the experimental results reported by Graybeal (2006, 2009) and Crane (2010), as shown in Table 7.3-3.
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\[ V_{cw} = f_t \left( 1 + \frac{f_{pc}}{f_t} b_w d_v \right) + V_p \]  \hspace{1cm} \text{Eq. 7.3-2}

where

\[ f_{pc} = \text{the compressive stress of the concrete at the centroid of the cross-section resisting external loads or at the junction of web and flange when the centroid lies within the flange.} \]

\[ V_s = \frac{A_v f_y d_v}{s} \cot \theta \]  \hspace{1cm} \text{Eq. 7.3-3}

\[ \cot \theta = \sqrt{1 + \frac{f_{pc}}{f_t}} \]  \hspace{1cm} \text{Eq. 7.3-4}

**Table 7.3-3. Comparison of Experimental Shear Capacities to Prediction Equations (Crane, 2010)**

<table>
<thead>
<tr>
<th>Specimen #</th>
<th>Failure Shear Force, ( V_{exp} ) (kip)</th>
<th>Predicted Shear Force by Direct Tension Approach, ( V_{n,DTS} ) (kip)</th>
<th>Predicted Shear Force by SMCF, ( V_{n,SMCF} ) (kip)</th>
<th>( V_{exp}/V_{n,DTS} )</th>
<th>( V_{exp}/V_{n,SMCF} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1 a*</td>
<td>452</td>
<td>279</td>
<td>376</td>
<td>1.62</td>
<td>1.2</td>
</tr>
<tr>
<td>1-2</td>
<td>431</td>
<td>235</td>
<td>287</td>
<td>1.83</td>
<td>1.5</td>
</tr>
<tr>
<td>2-1*</td>
<td>466</td>
<td>235</td>
<td>293</td>
<td>1.98</td>
<td>1.59</td>
</tr>
<tr>
<td>2-2</td>
<td>480</td>
<td>279</td>
<td>330</td>
<td>1.72</td>
<td>1.46</td>
</tr>
<tr>
<td>3-1*</td>
<td>422</td>
<td>243</td>
<td>287</td>
<td>1.73</td>
<td>1.47</td>
</tr>
<tr>
<td>3-2*</td>
<td>444</td>
<td>333</td>
<td>372</td>
<td>1.33</td>
<td>1.19</td>
</tr>
<tr>
<td>28S†</td>
<td>384</td>
<td>323</td>
<td>422</td>
<td>1.19</td>
<td>0.91</td>
</tr>
<tr>
<td>24S†</td>
<td>502</td>
<td>323</td>
<td>422</td>
<td>1.55</td>
<td>1.19</td>
</tr>
<tr>
<td>14S†</td>
<td>438</td>
<td>323</td>
<td>422</td>
<td>1.36</td>
<td>1.04</td>
</tr>
<tr>
<td>P2-21S‡</td>
<td>430</td>
<td>246</td>
<td>373</td>
<td>1.75</td>
<td>1.15</td>
</tr>
<tr>
<td>P4-57Sh‡</td>
<td>366</td>
<td>219</td>
<td>331</td>
<td>1.67</td>
<td>1.11</td>
</tr>
<tr>
<td>P4-57Ss‡</td>
<td>510</td>
<td>224</td>
<td>342</td>
<td>2.28</td>
<td>1.49</td>
</tr>
</tbody>
</table>

*Test failed in flexural compression.
†Graybeal (2006)
‡Graybeal (2009)

The direct tension method presented by Crane (2010) did not have as good a correlation with test results for his UHPC specimens as SMCF. The principal stress equation (Eq. 7.3-2) would be reduced to Eq. 7.3-5 when prestress is taken equal to zero:

\[ V_{cw} = f_t b_w d_v + V_p = 1.4 \text{ (ksi)} b_w d_v + V_p \]  \hspace{1cm} \text{Eq. 7.3-5}

Similarly, the angle \( \theta \) reduces to 45\(^\circ\) when prestress is zero. These two assumptions have been shown to be inaccurate when full fiber interaction in the member is accounted for. Crane concluded that the SMCF appeared to give closer predictions of the test results than the direct tension method.

Voo et al. (2010) evaluated the shear strength of UHPC I-Beams without stirrups. He included discussion of test results conducted by Voo et al. (2006) and by others. Voo et al., compared the experimental results with shear prediction theory based on Shear Plasticity Model (SPM) introduced by Johansen (1958), Sandbye (1965) and Nielsen (1967), and further developed by Zhang (1994) in defining a crack sliding model and by Voo et al. (2004) in developing the Variable Engagement Model (VEM).
The research specimens by Voo et al. were I-beam shaped, see Figures 7.3-3 and 7.3-4. The shear span was 2.0 m (6.6 ft) and the shear span-to-depth ratio varied from 2.4 to 4.5 with one specimen having a ratio of 1.8. That one specimen with a very small span-to-depth ratio behaved differently as will be discussed later. It proved to be unsuitable for vertical shear testing. The sudden flange/web transition is indicating, as it did in Crane’s study, that is makes the section more vulnerable to horizontal shear and/or vertical tension cracking.

Voo et al. (2006) tested seven specimens of the cross section shown in Figure 7.3-3 and with a length of 4.50 m (14.8 ft). Steel fibers were 2.5 percent of the concrete volume except for specimen SB4. Two fiber types were used: straight 13/0.2/1800 (length (mm)/diameter (mm)/tensile strength (MPa)) and end hooked 30/0.5/1000. The beams had a total of eighteen 0.5-in. diameter strands. Prestressing force was variable as shown in Table 7.3-4. The average compressive strength was 162 MPa (23.49 ksi) and flexural strength was 24 MPa (3.48 ksi). It is noted that the beam with the larger size, lower strength fibers (SB6) had a flexural strength of 25.2 MPa and apparently did not suffer reduction of concrete flexural strength compared to the concrete with the smaller, stronger fibers. The test series involved three prestress levels: no prestress, full prestress with an average value of P/A equal to 14.3 MPa (2.07 ksi), and 50% prestress with an average value of 7.15 MPa (1.04 ksi). Table 7.3-4 also shows the experimental ultimate shear force, $V_{exp}$, and angle, $\theta$, of the main crack near failure. If one uses an average shear stress at failure, $\tau_{avg}$, defined by $V_{exp}/(bw)(0.8h)$, the resulting values are shown in the table with average value of 2.26 ksi (Voo et al., 2006). The main crack angles are also shown in the table.

It is interesting to note that the angle was flatter than the 45 degrees. For members without prestressing and without fibers, ACI 318 and the AASHTO simplified method assume a crack angle of 45 degrees. In contrast, the AASHTO general method is based on the Modified Compression Field Theory in which the crack angle is based on...
strain and typically varies from 29 to 45 degrees for both prestressed and non-prestressed members. Also, fiber type did not seem to have a conclusive effect on the results. Voo et al. found out that their Shear Plasticity Model (SPM) gave predictions with an average theoretical/experimental value of 0.85 and a coefficient of variation of 0.079, which is conservative and consistent, especially in shear studies.

The study by Voo et al. in 2010 was a follow up to that of 2006. It also included results of the previous work as well as those from Hegger et al. (2004) and (2008). Voo et al. decided in the later series of tests to use a symmetrical I-beam section (see Figure 7.3.4) with six 0.5-in. strands in each flange. The beams were 8.6 m long to allow a study of the span-to-depth ratio. They assumed the effective depth to be (650 - 30) = 620 mm where the 30 mm is the cover to the centerline of the strands. The shear span-to-depth ratio varied from 1.8 to 4.5.

The second main variable in this study was the type and quantity of fibers. The fibers were three types: 0.2/15/2300, 0.2/20/2300 and 0.2/25/2300. They all had a volume ratio content of 1% (or 80 kg/m³), except specimen X-B7 which had 1.5% or (118 kg/m³). The compressive strength averaged 20.9 ksi and the flexural tensile strength averaged 1.6 ksi. Table 7.3.4 shows the main parameters and the shear failure results. The corresponding shear strength is 1.84, 1.98, 1.34, 2.54, 2.18, 2.91 and 3.25 ksi. It was noted in the paper that the results for beam X-B8 should be discarded due to the very low \( a/d \) value which resulted in longitudinal shear cracking between the web and the top flange. The shear span-to-depth ratio did not seem to have a pronounced impact as it varied from 2.5 in X-B4 to 4.5 in X-B6. As expected, an increase in fiber quantity corresponded to an increase in shear strength. Again, Voo et al. showed a good correlation with their SPM. A simple way to explain that model is given in Figure 7.3.5.
Through a series of iterations, the cracking angle, $\theta$, is determined. Compared to other methods, the predicted main crack angle appears to be relatively small. Once the angle is determined, the shear force capacity $V_n$ is determined based on equilibrium in a free body diagram as shown in Figure 7.3-5. A sum of the moments about point A results in the following relationship:

$$V_n = \frac{(f_t^*) (b_w) (x_d / 2) + (P_e) (d_p)}{a}$$

Eq. 7.3-6

Where, $a$ is the shear span, $f_t^*$ is the modified tensile strength of the concrete, $b_w$ is the web width, and $P_e$ is the effective prestress. The other symbols are dimensions defined in Figure 7.3-5.

The value of $f_t^*$ is proposed by Voo et al. (2010) was determined from Eq. 7.3-7. The tensile strength, $f_t$, is reduced by a tensile effectiveness factor of 0.80 (Voo, 2004) producing the effective tensile strength, $f_t^*$. The tensile strength of the matrix is affected by a number of factors including, global fiber orientation factor, volumetric fraction of fibers, bond strength of fibers as a function of fiber type and concrete strength. An average value of $f_t^*$ in Voo’s study is 4.3 MPa (0.63 ksi) and an average crack angle is 29 degrees.

$$f_t^* = v_t K_{f,\text{max}} \alpha_f \rho_f \tau_b$$

Eq. 7.3-7

where

- $v_t$ = tensile effectiveness factor = 0.80
- $K_{f,\text{max}}$ = maximum value of the global orientation factor = $0.5 - \frac{0.645}{\alpha_f^{9.55}}$
- $\alpha_f$ = aspect ratio of the fiber = $l_f / d_f$
- $\rho_f$ = volumetric fraction of fibers
- $\tau_b$ = bond stress between the fibers and the concrete matrix
- $0.23 \sqrt{f_t^*}$ (in ksi) for straight fibers

For the materials used in this PCI research, the corresponding $f_t^*$ values are 0.36 ksi for the 12 mm long fibers and 0.64 ksi for the 20 mm fibers. These values have been obtained based on the fiber properties and the design factors proposed by Voo. The method represented by Eq. 7.3-6 will be used in Phase II as a mean of comparing actual capacity to prediction for the test specimens. It will not be used as a design method for actual member design.

Pansuk et al. (2017) tested 6 I-beams of the cross-section dimensions shown in Figure 7.3-6. The main variables were fiber content and presence of shear reinforcement. The residual tensile strength was determined as 0.64 and 0.94 ksi for UHPC with 0.8 and 1.6% fiber content, respectively. The test results were compared to AFGC (2013) recommendations and with the fib Model Code (2010). The average reported shear stress at failure was 1.90 ksi, for specimens with fiber content of 1.6% by volume, using the web width and $0.9(0.9)h$ for shear effective depth. The predicted shear strength by both AFGC (2013) and Model Code (2010), using a cracking angle of 30°, was less...
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than one-half of the experimental capacity, with the Model Code providing a higher margin. It was also observed that shear reinforcement slightly contributed to the shear capacity, but also interfered with the flow of fibers in the molds. Presence of longitudinal reinforcement was shown by the authors to be important in creating a successful shear resistance model in the beams.

Figure 7.3.6. Shear Test Specimen (Pansuk et al., 2017)

Note: dimensions are in mm, 1000 mm = 39.37 in.

Foster et al. (2018) published an important paper that was the culmination of nearly twenty years of research on vertical shear in UHPC members by professor Foster and his research associates at the University of New South Wales in Sydney, Australia. Interestingly, one of professor Foster’s former students is Dr. Voo, who went on and established perhaps the most productive commercial operation in design and construction of UHPC bridges. Foster et al. adopted the Modified Compression Field Theory (MCFT) as the most suitable prediction method. They cited several international recommendations as proof of the recent popularity of this method, including the fib Model Code (2010), the Canadian Code (2014), the Australian Standard (2017), and the French Recommendations (AFRGC) (2013). This is particularly significant in this research as it is being proposed to adapt the current AASHTO specifications which have been based on the MCFT since 1993.

Foster et al. (2018) proposed modifications of the MCFT to account for steel fibers in the concrete. They validated the proposal with results obtained from the testing of 184 steel fiber reinforced concrete (SFRC) specimens with and without stirrups. They related the tensile capacity of the material to the standard flexural tests similar to that of ASTM 1609, rather than the direct tension test proposed by some researchers but seldom used in design guidelines for conventional or SFRC members. Foster et al. developed an inverse model for determination of the post-cracking residual direct tensile strength from the standard flexural test. A conversion of the deflection in the flexural testing to crack width, at which residual tensile stress is determined, was adopted from Vandewalle and Dupont (2003), using the following formula:

\[ w = 4\delta(0.9h)/L, \]

where \( w \) is crack mouth opening displacement, \( \delta \) is midspan deflection, \( h \) is prism height and \( L \) is prism span. When the standard dimensions in ASTM 1609 are substituted, the relationship becomes: crack width \( w = 1.2\delta \).

An important confirmation of previous studies (Minelli et al., 2014, Zarrinpour et al., 2015, Hussein and Amleh, 2018) is that size effect is essentially eliminated when adequate reinforcement (fibers or stirrups) are provided across the cracks. Foster and Agarwal (2018) found out that the size effect may only be significant in SFRC with strain softening and without stirrups. It is negligible in SFRC and in UHPC that exhibit strain hardening.

Foster and Agarwal (2018) compared their prediction model with the results of 81 shear tests, 52 SFRC and 29 UHPC prestressed beams. They proposed Eq. 7.3-8 for the contribution of the fibers. The equation has a reduction factor of 0.8 to account for fiber orientation and dispersion variability. The residual tensile strength should be calculated at certain crack widths as expressed by Eq. 7.3-9. The longitudinal tensile strain, \( \varepsilon_L \), at mid-depth is determined in Eq. 7.3-10. The calculation of the main crack angle, \( \theta \), is similar to AASHTO provisions with one difference: a factor of 7000 in Eq. 7.3-11 rather than 3500 in AASHTO is in recognition that it is at mid-height of the section. Figure 7.3-7 shows the experimental-to-predicted shear strength ratio versus member depth. It shows
that the proposed model consistently predicts lower shear strength than the experimental value; the ratio averages 1.90.

\[ V_f = 0.8 f_{rr} b_u d_v \cot \theta \]  
Eq. 7.3-8

\[ w = \frac{7.9 + 39370 \varepsilon_s}{\cos \theta} \cdot 10^{-3} \geq \frac{4.9}{\cos \theta} \cdot 10^{-3} \text{in.} \]  
Eq. 7.3-9

\[ \varepsilon_s = \frac{M_u}{d_v} + 0.5 N_u + 0.5 V_u \cot \theta - A_{ps} f_{po} \]  
Eq. 7.3-10

\[ \theta = 29^\circ + 7000 \varepsilon_s, \text{where } 20^\circ \leq \theta \leq 45^\circ \]  
Eq. 7.3-11

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In summary, previous research has demonstrated excellent vertical shear performance of UHPC members with fibers. Most tests have been conducted on pretensioned members without conventional shear reinforcement. For such members, researchers recommend considering separate concrete and fiber contributions to shear strength:

\[ V_n = V_c + V_f. \]  

The concrete contribution and crack angle are derived from the Modified Compression Field Theory. The fiber contribution to shear strength is based on residual tensile strength of the fiber reinforced composite across the inclined crack. The residual tensile strength is determined from flexural or uniaxial tension tests of the fiber-reinforced UHPC and adjusted for fiber orientation and other factors. These commonalities in the research are also reflected in international practices, as described in the next section.

### 7.4 International Practices

The Association Française de Génie Civil (AFGC) (2013), called herein the French Recommendations, provides guidelines for shear design of UHPC members. The contribution to shear strength of concrete \( V_c \), fibers \( V_f \), and steel stirrups \( V_s \) is given by the following equations.

\[ V_n = V_c + V_f + V_s \]  
Eq. 7.4-1

where

\[ V_n = \text{nominal shear resistance} \]
\[ V_c = \text{nominal shear resistance of the concrete} \]
\[ V_f = \text{shear resistance provided by fibers} \]
\[ V_s = \text{shear resistance provided by transverse reinforcement} \]

The formula for fiber contribution:
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\[ V_f = \left( \frac{f_{rr}}{K} \right) b_v d_v \cot \theta \]  

where \( f_{rr} \) = post-cracking residual tensile strength of the fiber-reinforced cross-section \( K \) = fiber orientation factor = 1.0-1.4 \( b_v \) = effective web width \( d_v \) = effective shear depth \( \theta \) = angle of inclination of diagonal compressive stresses, also called main crack and diagonal strut, not to be taken less than 30°

The residual tensile strength is calculated as the area under the stress-strain diagram after cracking divided by the difference between the strain at cracking and the ultimate strain. The nominal shear resistance shall not be taken greater than the ultimate strength of the compression strut as given in Eq. 7.4-3. The equation is the US version with \( f_{rr} \) in ksi. Note that AFGC (2013) recommendations include partial safety factors, which are not parallel to the resistance (strength reduction) factors used in US practice.

\[ V_n \leq \frac{f_{rr}^{2/3}}{(\cot \theta + \tan \theta)} b_v d_v \]  

Eq. 7.4-3

AFGC (2013) allows for design with no steel stirrups as long as steel fibers are used with certain minimum requirements.

The Swiss Standard (2016) (SIA 2052, 2016), ignores the plain concrete contribution and accounts for fiber contribution into one term, equal to \( b_v d_v f_{rr} \cot \theta \), which is similar to the French Recommendations. The value of \( f_{rr} \) in the Swiss Standard is the average of the first peak cracking strength and the ultimate strength in tension. The strut angle, \( \theta \), is taken not less than 30 degrees for “normal” cases, 25 degrees for cases with significant axial compression (i.e. prestressed), and 40 degrees for cases with significant axial tension. The Swiss Standard specifies minimum tension tie at the support and maximum shear resistance, similar to the provisions given in AASHTO. However, the maximum shear is allowed to be as high as 0.55 \( f_c \), which is considerably higher than the 0.25 \( f_c \) limit in AASHTO.

The Japan Society of Civil Engineers (JSCE, 2008) has developed recommendations to calculate shear strength for the design of UHPC similar to Eq. 7.4-1. JSCE (2008) added an additional term to account for the vertical component of harped prestressing strands. The residual tensile strength in Eq. 7.4-2 was conservatively replaced with design tensile yield strength. The angle of inclination was assumed as 45 degrees. Eq. 7.4-3 is replaced with Eq. 7.4-4, where \( f_c \) is in ksi. It was reported that these recommendations are conservative even in the case of the elimination of shear stirrups.

\[ V_n \leq 0.48 \sqrt{f_c} b_v d_v \]  

Eq. 7.4-4

The Australian practice appears to be moving in the direction of adopting the Modified Compression Field Theory as used in AASHTO and Canadian codes. A representation of this direction is given in Foster and Agarwal (2018), which is discussed in the preceding section.

Annex 8.1 (2018) of the Canadian Highway Bridge Design Code (CSA S6-14) was released for public review at the end of 2018. It presents the MCFT as an efficient approach to predict the shear strength of fiber reinforced concrete, FRC. The Annex shows, according to Foster et al. (2018), separate terms for the contributions of concrete, fibers, stirrups, and the vertical component of prestress as shown in Eq. 7.4-5. The contributions of concrete, stirrups, and prestressing are same as specified in the main body of the Canadian Highway Bridge Design Code for conventional concrete. The fiber contribution to shear strength is per the Foster et al. (2018) approach. It should be noted that the Canadian Annex utilizes additional partial safety factors to reduce the nominal strength, which is unlike American codes and specifications for structural design. Eq. 7.4-6 presents the fiber contribution, \( V_f \), to shear strength as specified by Annex 8.1 (2018). The fiber efficiency, \( \gamma_f \), is specified as 0.40 for large structural components such as footings, beams, and girders. The specified post-cracking tensile strength, \( f_{rr} \), of FRC should be determined corresponding to a crack opening, w. The crack width is calculated in accordance with Foster and
Agarwal (2018) for SFRC with strain softening as shown in Eq. 7.4-7. The equivalent crack spacing parameter, $s_{xe}$, is influenced by aggregate size, $a_g$, and calculated per Eq. 7.4-8. The aggregate size for UHPC should be taken as zero. The crack spacing parameter, $s_a$, is taken as the lesser of either $d_v$ or the maximum distance between layers of longitudinal crack control reinforcement, where the area of the reinforcement in each layer is not less than 0.003$b_0s_a$. The net longitudinal tensile strain in the section at the centroid of the tension reinforcement, $\varepsilon_s$, shall be calculated in accordance with Eq. 7.4-9. For SFRC with strain hardening and UHPC, Foster and Agarwal (2018) dropped the size effect by setting the second term in Eq. 7.4-7 and Eq. 7.4-10 equal to 1.0.

$$V_n = V_c + V_f + V_s + V_p$$  \hspace{1cm} \text{Eq. 7.4-5}

where

$$V_p = \text{component of prestressing force in the direction of the shear force (kip)}$$

$$V_f = 0.8f_r f_{y,v} d_v \cot \theta$$  \hspace{1cm} \text{Eq. 7.4-6}

$$w = \left( \frac{7.9 + 39.370\varepsilon_s}{\cos \theta} \right) \left( \frac{39.4 + s_{xe}}{51.2} \right) (10^{-2}) \geq 0.005 \text{ in.}$$  \hspace{1cm} \text{Eq. 7.4-7}

$$s_{xe} = \frac{1.38}{s_x a_g + 0.63}$$  \hspace{1cm} \text{Eq. 7.4-8}

$$\varepsilon_s = \frac{M_u}{d_v^2} + 0.5N_u + (V_u - V_p) - A_{ps f_p} \frac{2(E_s A_s + E_x A_s)}{2(E_s A_s + E_x A_s)}$$  \hspace{1cm} \text{Eq. 7.4-9}

$$\theta = (29^\circ + 7000\varepsilon_s)(0.88 + \frac{s_{xe}}{100})$$  \hspace{1cm} \text{Eq. 7.4-10}

The shear strength is limited to Eq. 7.4-11 and Eq. 7.4-12 for members with and without continuous flexural reinforcement, respectively. Eq. 7.4-11 is the same as AASHTO requirements for conventional concrete without fibers. However, these limits may not control for members with fibers and without conventional shear reinforcement because the fiber contribution is not allowed to exceed the concrete contribution, which severely limits the allowable shear strength compared to other international standards. This might be attributed to the fact that the Canadian Annex proposes general specifications for all types of fiber reinforced concrete.

$$V_n \leq 0.25 f'_{c'} b_v d_v + V_p$$  \hspace{1cm} \text{Eq. 7.4-11}

$$V_n \leq \text{Min.} \left(0.04 \sqrt{f'_{c'}} b_v d_v, 0.25 f_{yr} b_v d_v \right)$$  \hspace{1cm} \text{Eq. 7.4-12}

Neither ACI 318-14, nor the yet to be published ACI 318-19, currently covers UHPC. One exception is that Steel Fiber Reinforced Concrete (SFRC) is allowed to be used to eliminate the requirement for minimum shear reinforcement. According to ACI 318-14, Sec. 9.6.3.1, the minimum shear reinforcement is not required for steel fiber-reinforced normal weight concrete conforming to Sec. 26.4.1.5.1(a), Sec. 26.4.2.2(d), and Sec. 26.12.5.1. The fibers shall be steel fiber, deformed, conform to ASTM A820, and have $l/d_v$ of at least 50 but not more than 100. The fiber content shall be at least 100 pounds per cubic yard of concrete, or 0.75% by volume. The residual strength, obtained from flexural testing in accordance with ASTM C1609, at a midspan deflection of span/300 is not less than 90% of the measured first-peak strength, nor 90% of the strength corresponding to 0.24$\sqrt{f'_{c'}}$. The same residual strength midspan deflection of span/150 is not less than 75% of the measured first-peak strength nor 75% of the strength corresponding to 0.24$\sqrt{f'_{c'}}$.

It should be noted that the concrete specified in this program exceeds the conditions given above, except that the fibers are high strength small diameter straight bars with a volume content of 2%. An additional condition placed in this project is that the minimum first-peak cracking strength is 1.50 ksi and the ultimate cracking strength is 2.00 ksi in the ASTM C1609 test, ensuring high tensile strength and adequate strain hardening load-deflection behavior. Therefore, there does not seem to be any need for stirrups as long as the concrete and fibers meet the minimum material behavior requirements and are capable of resisting the applied shear force.
7.5 Proposed Tentative Design Recommendations

It is proposed that shear design be performed with the following procedure which is an adaptation of the current AASHTO Modified Compression Field Theory (MCFT). It appears from the extensive literature search that this method is gaining considerable acceptance worldwide, as indicated in the preceding sections.

The primary check is the strength limit state due to factored loads. As will be seen in the discussion below and in the numerical examples, there appears to be considerable capacity at the critical section, assumed to be at \( d_e \) from the face of the support, where \( d_e \) is the effective shear depth. Once that section is satisfied, the capacity-demand margin in the remainder of the beam increases. It is shown that the capacity at the critical section is significantly higher than the demand without need for steel stirrups in the cases considered so far. Therefore, it is expected that no stirrups will be required in the design of UHPC members meeting the minimum materials requirements of this research.

A second check that must be performed is the service limit state as member spans get relatively large and their webs get relatively small. Again, the AASHTO specifications will be used for this check. The stress under service dead loads plus live loads, plus the effect of prestress, is to be kept to a limit below that causing diagonal cracking. It is possible to allow diagonal cracking due to service loads when UHPC is used due to the very high toughness and fatigue limits of this material. However, at this stage of implementation, questions still remain about formulas for cracking and fatigue limits that might inhibit rapid mobilization of this outstanding material. So, it is conservative at this time to design the members to be uncracked under service loads. Such conservatism does not seem to cause the resulting systems to be uneconomical.

The sections below illustrate the steps required to be followed to apply the proposed design recommendations.

7.5.1 Strength Limit State

The load factor demands, \( V_s \) and \( M_w \), at the critical section are calculated in the typical manner, using available commercial software. The location of the critical section is at \( d_e \) away from the support face. The effective depth \( d_e \) is defined per AASHTO as the largest of three quantities: the distance between the tension and compression resultant forces due to flexure, \( 0.9 \, d_v \) or \( 0.72 \, h \), where \( d_v \) is the effective depth in flexure, and \( h \) is the total depth.

The demand is checked against the design capacity, reduced by a resistance factor \( \phi = 0.9 \). The nominal capacity (resistance), \( V_n \) without the \( \phi \)-factor, is calculated from Eq. 7.5.1-1. Eq. 7.5.1-1 is the same as Eq. 5.7.3.3-1 of AASHTO except that a component related to fiber resistance, \( V_f \), is added here. The other components, \( V_o, V_p \) and \( V_p \) are calculated in the same manner as in AASHTO. To get the value of \( V_c \), the coefficient \( \beta \) is calculated to reflect the effects of the longitudinal strain, \( \varepsilon_v \), in the member at the level of the longitudinal tensile reinforcement at the section being considered. This proposed approach uses the simplified \( \beta \) formula in AASHTO (Eq. 5.7.3.4.2-1), which is reproduced here as Eq. 7.5.1-3. The second equation (Eq. 5.7.3.4.2-2) includes a factor for size effect, which recognizes that sections that do not contain at least the minimum amount of stirrups can exhibit reduction of capacity as depth increases. That equation does not apply here as there is no presence of fibers in the concrete mix compared for any need for stirrups.

The strain, \( \varepsilon_d \), is calculated using AASHTO Eq. 5.7.3.4.2-4 (Eq. 7.5.1-4). The tension strain is large when there is little or no prestressing. The tension strain also increases with an increasing value of the applied moment (away from the support). Thus, at the critical section for prestressed members, the strain, \( \varepsilon_d \), is generally small or negative, in which case it defaults to 0. The corresponding value of \( \beta \) defaults to 4.8. Substituting 18 ksi for concrete strength, Eq. 7.5.1-2 reduces to \( V_c = 0.64 \, b_v \, d_v \). For reinforced (non-prestressed) concrete members the value of \( \beta \) may be much smaller.

The longitudinal strain, \( \varepsilon_d \), is also used in Eq. 7.5.1-4 (AASHTO Eq. 5.7.3.4.2-3) to calculate the angle of inclination of diagonal compression strut, \( \theta \). This angle also indicates the direction of the main diagonal cracking. The angle, \( \theta \), is an important parameter in determining the effectiveness of the fibers, the flatter the angle, the more fiber reinforced area is engaged in resisting vertical shear. Tests from the literature seem to indicate that an angle of 30 degrees is quite common and an angle near 45 degrees is rare. Note that 45 degrees is the crack angle typically assumed for non-prestressed conventionally reinforced sections without steel fibers but with steel stirrups in the ACI 318 and in some of the AASHTO approximate procedures. This is especially true when ACI 318 Code is used for design of building members. The method proposed here is intended to be used to predict the capacity in shear whether the member is prestressed or non-prestressed. Thus, it will be interesting in the testing program in Phase
To observe the main crack angle for members without prestressing. It is expected that the fibers will have some impact on the values of $\varepsilon_s$ and $\theta$, and that refinements of the formulas for these parameters may be included.

The most significant contribution to shear strength is the fiber factor, $V_f$. Eq. 7.5.1-5 is similar to that in the French Recommendations (AFGC, 2013), Eq. 7.4-2. The value of the residual tensile strength, $f_r$, is taken in the French Recommendations as the average of the value at first peak cracking and the ultimate value, with a factor for fiber orientation included. In the proposed method a value of $f_r$ of 1.0 ksi may be used. Its selection is justified on the basis of the collection of test results as given in the preceding sections, and on international codes and recommendations, especially the French and Korean ones. For example, AFGC formula for $V_f$ given by Eq. 7.4-2, used ($f_r/K$), where $f_r$ is the post-cracking residual strength and K is the fiber orientation factor. A conservative estimate of this ratio is 1.31/1.25 where the 1.31 ksi is a lower bound of the values observed in testing in France and also in agreement with results by Graybeal (2006) and Crane (2010). The fiber orientation factor varies from 1.0 for favorable orientation to 1.4 for unfavorable orientation with 1.25 used for random orientation in the Korean guidelines (2012). It has been the desire in this project not to inhibit the precasters from placing the concrete in a certain way, but rather advise of ways to have favorable orientation caused by good placement practices. However, there would be no penalty for other methods of placement that the precasters see as most efficient for their production methods. Therefore, it would be reasonable to assume in design random fiber orientation in the precast product, thus the 1.25 factor. The ratio is 1.31/1.25 = 1.04 ksi. Conservatively, the residual tensile strength for shear design, $\psi f_{fu}$ is tentatively used as 0.75 ksi in the proposed design procedure. Because it is specified in this project that the minimum specified value of the peak strength, $f_{0u}$ in the ASTM C1609 test is 2 ksi, the limit being proposed may be stated in a general way to be $= 0.375 f_{fu}$. The $\psi$ value is a conversion factor from flexural bending strength to post-cracking tensile strength, which is specified as 0.383 and 0.37 by the Swiss Standard (SIA 2052, 2016) and German Guideline (DAfStb, 2017), respectively. In Phase II, $\psi f_{fu}$ will be further examined and refined based on statistical analysis of testing by our team and also previous work.

The research reports and international standards described previously, as well as the proposed tentative design recommendations, assume that the fiber contribution to shear strength is proportional to $b_n d_w$. However, tension in fibers across the portions of diagonal cracks that extend through the flanges may also contribute to shear strength, especially where large radii between the webs and flanges are used. This potential effect will be evaluated in Phase II.

The shear check is not complete without the all-important tension tie requirement. It is important to extend the flexural reinforcement into the end of the member on the support side of the crack such that a minimum amount of tension tie capacity is reached, (AASHTO Equation 5.7.3.5-2). This is a critical check, especially for building members bearing on a relatively narrow beam ledge of a bearing wall. Without an adequate tension tie, it is often observed in shear testing to actually result in bond (or strand slippage) primary failure, which may be followed with diagonal cracking. It should be noted that AASHTO Equation 5.7.3.5-2 should be applied at the inside edge of the bearing area of simple end supports. The term of $\frac{|M_{ul}|}{d_v \phi_f}$ should be added to the demand of other sections.

$$V_n = V_c + V_f + V_p + V_p$$  \hspace{1cm} Eq. 7.5.1-1

$$V_c = 0.0316\beta \sqrt{f'_c} b_n d_v$$  \hspace{1cm} Eq. 7.5.1-2

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)}$$  \hspace{1cm} Eq. 7.5.1-3

$$\varepsilon_s = \frac{(M_u/d_w) + (V_u - V_p) - \phi_f}{(E_S A_S + E_P A_P)} \geq 0.0$$  \hspace{1cm} Eq. 7.5.1-4

$$V_f = \psi f_{fu} b_n d_v \cot \theta$$  \hspace{1cm} Eq. 7.5.1-5

$$\theta = 29 + 3,500 \varepsilon_s$$  \hspace{1cm} Eq. 7.5.1-6

$$A_s f_y + A_p f_p s \geq \left(\frac{V_c}{\phi_f} - 0.5V_c - V_p\right) \cot \theta$$  \hspace{1cm} Eq. 7.5.1-7

Eq. 7.5.1-7 is based on moment equilibrium about the compression zone, assuming a diagonal crack at angle $\theta$ extending from the support. The $V_c$ term, which reduces tension tie demand, would not apply to UHPC designs.
without conventional shear reinforcement. However, fiber tension across the diagonal crack should similarly reduce tension tie demand. This effect will be explored in Phase II.

The maximum allowed nominal shear resistance in a section is given by AASHTO Equation 5.7.3.3-2, \( V_n = 0.25 f'_c b_n h + V_p \). This is much larger than that allowed by ACI 318 as ACI 318 does not require a minimum tension tie at the member end and thus must allow for the reduced capacity resulting from a lack of an adequate tie. The AASHTO equation is similar to that specified by the Canadian Annex 8.1 (CSA-S6 Equation A8.1.9.2). AASHTO Art. 5.7.3.2 indicates that the 0.25 \( f'_c \) limit should be reduced to 0.18 \( f'_c \) for beams not built with strands anchored into cast-in-place diaphragms. It is proposed in this research to use the lower limit, Eq. 7.5.1-8, for simplicity since it will not be exceeded in the great majority of applications. A comparison between the proposed limit and AFGC (2013) for \( f'_c \) of 18 ksi, with \( \theta \) assumed equal to 35 degrees, shows comparable values of 3.24 and 3.25 ksi. The corresponding ACI 318 limit would be much lower, about 1.61 ksi.

\[
V_n \leq 0.18 f'_c b_n h + V_p \tag{Eq. 7.5.1-8}
\]

No minimum shear reinforcement is needed since the fibers already contribute a relatively large amount of shear capacity.

ACI 318-14 cannot be used directly for design of UHPC and must be significantly modified before it is used. In the examples below, we are proposing that ACI 318-14 be modified in two aspects. First, the current restriction that \( f'_c \) to not be taken larger than 10 ksi in shear design should be extended to 18 ksi. ACI 318-14, Article 22.5.3.2 for members with the minimum number of stirrups is currently exempt from the 10 ksi limit. With fibers as specified in the mixes used in this project, there should be no restriction. Second, the angle of the main shear crack (compression strut) should not be set at a fixed quantity of 45 degrees, but replaced with a provision that allows it to range from about 29 to 45 degrees, depending on fiber content, level of prestress and possibly developed flexural reinforcement content in the section being considered. Finally, a check on the adequacy of the tension tie across the main crack should be made, similar to what is in AASHTO, the Canadian and other international codes. This last point is critical in some precast prestressed concrete systems where the support does not allow for an extensive embedment of the flexural steel beyond the expected location of the main crack. In this situation, extra anchorage needs to be provided. It is comforting to know that strand bond in UHPC is excellent and bond length is relatively very short. However, it may not be enough when the bearing length is only 6 inches and the member has to end at the face of an inverted tee beam or a wall panel. For these reasons, the recommend procedure is as given above, while designers who prefer to use the ACI Code may do so with the modifications listed here.

### 7.5.2 Service Limit State

The conditions at service load levels should be such that there is no excessive diagonal tension cracking or fatigue concern (in the case of bridges). It is well known that fiber reinforced concrete has superior capacity in both criteria compared to conventional concrete with no fibers. However, at this stage of development, the research team proposes that principal tensile stress at service load levels be limited to a value below the cracking limit and therefore no diagonal tension cracking would be expected in UHPC members under service loads. Analysis may be done in conventional elasticity theory to determine the principal tensile stress in the web at the most critical point, which may be near the top of the web at the junction with the top flange near the end of the member. The stress limit in this case is tentatively chosen here to be 0.75 ksi until experimental results are available in Phase II. It is more conservative than the work done by Gowripalan and Gilbet (2000) in Australia for Ductal. In that report, they suggested that the principal stress be limited to 0.725 + 0.05\( \sqrt{f'_c} \) = 0.94 ksi, for 18 ksi concrete. It is also more conservative than the recommendation by Wipf et al. (2009) in their design of the Buchanan County Bridge, the first UHPC bridge in the United States in the state of Iowa. Their work, sponsored by FHWA, proposed a cracking strength of 1.1 ksi for the UHPC used for the bridge which had a compressive strength of 24 to 25 ksi.

### 7.6 Finite Element Modeling of Vertical Shear Specimens Planned for Phase II

The finite element analysis method (FEM) allows the research team to examine the behavior of UHPC beams as the load increases to failure levels and to bring the test program to high focus. This gives the research team an opportunity to detect areas of distress in the shear testing program as the loading is applied. It is also hoped that such a tool will be useful in optimizing the testing program and extrapolating for cases not tested. The FEM was conducted using ANSYS software which allowed for material non-linearity.
The UHPC was modeled utilizing 3D solid element, SOLID185, with 8 nodes and three degrees of freedom at each node. SOLID185 has plasticity, hyper-elasticity, stress stiffening, creep, large deflection, and large strain capabilities. It has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyper-elastic materials. The bearing pads were modeled utilizing SOLID185 as well. The continuous reinforcement was modeled utilizing 3D link element, Link180, with 2 nodes and three degrees of freedom at each node. Link180 is a uniaxial tension-compression element.

The FEM results were initially calibrated to existing data from Chen and Graybeal (2012) to validate the performance of the model. Chen and Graybeal employed concrete damage plasticity model which is demonstrated to be superior to concrete smeared cracking model in terms of replicating physical test results. They studied the flexural and shear performance of UHPC AASHTO Type II prestressed I-girders. Test 80F and Test 24S, in Chen and Graybeal, were utilized to calibrate the proposed ANSYS model in flexure and shear, respectively. UHPC was modeled with unit weight, modulus of elasticity, and Poisson's ratio of 160 lb/ft³, 8000 ksi, and 0.18, respectively. **Figure 7.6-1** presents the assumed uniaxial stress-strain relationship. The tensile behavior was modeled as a bilinear diagram with limit of elasticity of 2.3 ksi (15.9 MPa) corresponding to strain of 0.0084 and assumed ultimate tensile plastic strain of 0.01. The ultimate compressive stress and strain were modeled as 28 ksi (193 MPa) and 0.004, respectively. **Table 7.6-1** compares the results conducted from ANSYS model and those reported experimentally and analytically by Chen and Graybeal. It shows that analytical model results are in a good agreement with the analytical results by Chen and Graybeal.

![Figure 7.6-1. Conceptual UHPC Uniaxial Stress-Strain Relationship (Chen and Graybeal, 2012)](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>ANSYS Model</th>
<th>Chen and Graybeal (2012)</th>
</tr>
</thead>
<tbody>
<tr>
<td>80F</td>
<td>Ultimate Load, P_u</td>
<td>172 kips</td>
</tr>
<tr>
<td></td>
<td>Deflection @ P_u at the load location</td>
<td>19.3 in.</td>
</tr>
<tr>
<td>24S</td>
<td>Ultimate Load, P_u</td>
<td>617 kips</td>
</tr>
<tr>
<td></td>
<td>Deflection @ P_u at the load Location</td>
<td>1.0 in.</td>
</tr>
</tbody>
</table>

ANSYS model was further modified to incorporate the characteristics of the UHPC properties specified in this research project. The goal was to investigate the parameters affecting shear behavior of the UHPC in the testing program proposed to be conducted in Phase II of the project. In the model, UHPC was modeled with density, modulus of elasticity, and Poisson's ratio of 155 lb/ft³, 6500 ksi, and 0.2, respectively. **Figure 7.6-2** presents the assumed UHPC uniaxial stress-strain relationship. The tensile behavior was modeled as a trilinear diagram with limit of elasticity of 2.0 ksi corresponding to a strain of 0.0003. The tensile stress was assumed constant for strain up to 0.004. Beyond this limit, the stress is decreases linearly to zero at 0.01 strain. The ultimate compressive stress and strain were modeled as 18.0 ksi and 0.004, respectively.
To avoid stress concentration problems at supports and applied loads, a 3D solid element with elastic material properties is used as a bearing device. The material is assumed to be steel elasticity modulus of 28,935 ksf and Poisson’s ratio of 0.3. A contact element having a friction coefficient of 0.2 was inserted between the concrete beam and bearing plate. The stress–strain relationship of the prestressing strands and mild steel reinforcement were defined using the power formula, Eq. 7.6-1. Table 7.6-2 presents the factors used in the power formula for different types of reinforcements. ASTM A1035 mild steel reinforcement, grade 100 ksi was used as flexural reinforcement in addition to the strands ensure that shear failure would take place before flexural capacity is reached, and to examine the effects of various prestressing levels.

$$f_s = E_s \varepsilon_s \left[ Q + \frac{1 - Q}{\left(\varepsilon_s + \frac{f_{ps}}{f_{pu}}\right)^{1/R}} \right] \leq f_{pu}$$

\textbf{Eq. 7.6-1}

**Table 7.6-2. Stress-Strain Power Formula Factors for Steel Reinforcement**

<table>
<thead>
<tr>
<th>Steel Type</th>
<th>$f_{pu}$ (ksi)</th>
<th>$E_s$ (ksi)</th>
<th>$Q$</th>
<th>$f_{ps}$ (ksi)</th>
<th>$R$</th>
<th>$K$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM A1035</td>
<td>150</td>
<td>29000</td>
<td>0.035</td>
<td>140</td>
<td>2.042</td>
<td>1.02</td>
</tr>
<tr>
<td>Strand</td>
<td>270</td>
<td>28500</td>
<td>0.031</td>
<td>243</td>
<td>7.36</td>
<td>1.04</td>
</tr>
</tbody>
</table>

Prestressing level, web thickness, beam size, and shear reinforcement are some of the parameters used in the analysis. **Figure 7.6-3** shows views for the generic model used in this study. It also shows the continuous flexural reinforcement template. All beams were for 1-beam with 2-ft overhang at each end beyond the supports. The overhang is provided to avoid slippage of the tensile reinforcement which would result in possible bond failure, reducing the shear capacity of the beams. The shear reinforcement, if any, was also ASTM A1035, using single legs spaced uniformly over the total length of the beam. The load was applied at the midspan and incrementally increased to failure.
A sample of the results is presented, in Figures 7.6-4 and 7.6-5. It shows the shear behavior and expected mode of failure of a 34 in. deep I-beam. The beams have 3-in. web and span 12 ft. They have no stirrups. The flexural reinforcement was (12) 0.5-in. strands and 12#6 Grade 100 ksi bars. The bars are #4 and are spaced at 10 in. The beam reached a peak load of 693 kips at a midspan deflection of 0.3 in. as shown in Figure 7.6-4. Figure 7.6-5 presents the principal stresses at the failure load of the beam. The principal stresses reached the ultimate tensile strength in the web near the top fillet. Strain softening began to develop after the peak stress as expected, decreasing the stress until ultimate strain is reached, signaling total failure. This is shown in Figure 7.6-5 in red color in the web stresses.
The strain compatibility approach and modified compression field theory were utilized to predict the flexure and shear failure loads, respectively, as proposed in this research. The ultimate tensile strength was assumed to be 2.0 ksi for flexure and shear prediction models, similar to the FEM assumptions. The predicted shear failure load was 609.0 kips, less than that predicted by the FEM, which demonstrates the conservative assumptions used in design. As the testing in Phase II is conducted, further FEM will be used and correlated to the behavior observed in the testing.

### 7.7 Design Example of Contribution of Fibers to Shear Strength

To evaluate the proposed procedure, this section shows a summary of the calculations for bridge example. The decked I-beam is considered one of the most efficient bridge product shapes to span 250 ft. The proposed beam has a 4-in. wide web and no steel stirrups. Figure 7.7-1 shows the factored load shear demand, according to AASHTO Strength I Limit State. The values in the figure are for $V_u/\phi$ for direct comparison with the nominal capacity, where
the resistance factor $\phi$ is 0.9. The nominal shear capacity according to the proposed method is also shown. Please note that the only two contributors to $V_n$ are the concrete, $V_c$, and the fibers, $V_f$.

For the first 50 feet of the span, both the concrete and fiber capacities remain constant. Beyond that point, flexural effects create significant tensile strains which in turn reduce the $\beta$ values and increase the cracking angle, $\theta$. As can be seen, the conservatively assumed contribution of the fiber shearing stress, $\psi f_{fu}$, equal to 0.75 ksi, creates a significantly higher capacity than the demand.

The figure also shows the results of design with ACI 318-14 without any modification and with the modifications suggested above. The current ACI Code, without modification, would result in predictions smaller than the demand and thus an unsafe design. Modifications to ACI include adding the same fiber resistance force as proposed for AASHTO and using a variable cracking angle as is already in AASHTO. Lastly, the limit on compressive strength not allowed to be taken greater than 10 ksi is removed. This figure shows that the proposed model predicts less capacity than the modified ACI, primarily due to the limit placed on $\beta$ used in predicting $V_c$. It is not allowed to exceed 4.8 in AASHTO.

![Comparison of Proposed Capacity versus ACI 318 and AASHTO LRFD](image)

Finally, it is to be observed that the upper limit on shear resistance, beyond which the member size needs to be increased, predicts compression strut failure by ACI while it is safe according to AASHTO, using the limit $0.18 f'_c b vd = 0.18(18)(4)(0.8)(108) = 1120$ kip. The AFGC Recommendations also show higher maximum allowed shear capacity than that provided in the beam. This issue will require more investigation in Phase II to confirm that no compression failure takes place in members stretched out to near their limits.

### 7.8 Plans for Phase II Testing

This research project is highly focused on providing adequate guidelines and examples for immediate implementation of a class of UHPC, defined in the project, to certain types of building and bridge members, developed in this project. It is not intended to focus on academic research to advance the understanding of fiber reinforced concrete behavior or to extend applications beyond the stated ones. The concrete specified in the project must have minimum specified compressive strength of 10 ksi at prestress release and 18 ksi at service. It must have post cracking peak flexural tensile strength of 1.5 ksi and ultimate (maximum) flexural tensile strength of 2 ksi, in addition to certain ductility requirements that ensure strain hardening in the standard ASTM C1609 flexural testing.

The products of interest in this project are plant produced precast prestressed concrete members that can span up to 250 ft for bridges and 60 ft for buildings. The bridge products are decked I beams, U beams and box beams. The building beams are voided inverted tee beams. And the building joists are thin walled voided slabs with stem openings (block-outs) to allow for placement of utilities within the structural depth. All these products are always
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Premixed concrete is capable of withstanding the applied loads even when reinforced with fibers, provided there is no shear reinforcement. However, few of the specimens will have stirrups as given below.

Therefore, any guidelines presented in this report are to be understood to be specific to the applications being considered, although much of the information will be beneficial to precast pretensioned UHPC applications. This approach was selected for this project in order to allow for rapid implementation of this outstanding material in optimization of actual structural members and not limit it to joints and overlays, as is dominant in current practice.

The design examples presented in this report and the discussion given in the preceding sections point to the observation that steel fibers of the type and quantity used here are capable of safely resisting the applied shear forces without additional help from steel bars. This situation is very favorable in design, detailing and production of precast concrete members where considerable time is spent in ensuring bars are designed and detailed properly and placed without violation of clear cover requirements. It is also beneficial when concrete contains fibers as the presence of vertical bars in narrow stems can disrupt the flow of the fibers. Further, member stems can be as narrow as 2 inches, which is a significant factor is reduction of member weight and concrete quantities of this relatively expensive material.

The proposed design method given in Section 8.5 indicates that the current AASHTO shear design provisions, General Method, appears to be applicable with minor modifications. This is very favorable as it allows the research team to add the least amount of revision to an established method. It is also reassuring to note that, with the exception of the ACI 318 Code, most of the major international codes and guidelines seem to support slightly varying versions of the same approach which is based on the Modified Compression Field Theory as expressed by Bentz et al (2006).

Basically, the resistance at any given cross section reduces to two forces, the force contributed by concrete alone without fibers, and that contributed by the fibers:

\[
V_c = 0.0316 \beta \sqrt{f'} b_v d_v \quad \text{Eq. 7.8-1}
\]

\[
V_f = \psi f_{fu} b_v d_v \cot \theta \quad \text{Eq. 7.8-2}
\]

As noted above, \( \psi f_{fu} \) is taken as 0.75 ksi

The concrete contribution is straightforward. The coefficient \( \beta \) has a narrow range, varying from about 2 to 4.8, depending on the axial strain in the longitudinal direction at the section being considered. However, we still do not know if this resistance force is altered significantly when fibers are added.

The fiber contribution is the more significant force that needs to be studied in detail and confirmed with full scale testing. The assumed \( \psi f_{fu} \) of 0.75 ksi and the angle \( \theta \) need to be studied in detail. The value of \( V_f \) can double if the angle is changed from 45 to 29 degrees. The angle is a function of the axial strain which is a function of level of prestress, amount of flexural reinforcement and relative values of flexure and shear in the section in question.

Previous research has shown this angle to be in the range of 25 to 40 degrees. It appears to be always smaller than 45 degrees, contrary to what is implied in the ACI Code, even when there is no prestress. It is also possible that the presence of the fibers causes this angle to be relatively shallow. But we must be sure through analysis and testing as shallow angle produces high resistance.

It has already been established in the literature and various codes that minimum stirrup reinforcement is not required in UHPC. Even, in ACI 318 the requirement is waived for SFRC of conventional strength when certain fiber conditions and content are met. There may be a concern that size effect may alter the shear resistance of the concrete component. But this concern is only limited to concrete without shear reinforcement, whether it is fibers or rebar.

Fiber orientation is an important factor in ensuring that the design value for the tensile strength of the concrete contributing to shear, \( \psi f_{fu} \) which is taken as 0.75 ksi in Eq. 7.8-2, and considered to be reliable. As would be expected, UHPC is more difficult to place and consolidate in the steel forms especially in narrow stems and thin bottom flanges. Attempting to force the fibers to be aligned in a way that gives the most structurally beneficial effect is futile. For example, horizontal fiber alignment at midspan and vertical fiber alignment at the critical shear area...
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would be beneficial. However, neither alignment can be guaranteed; design must account for the randomness of the fiber orientation with adequate factors of safety. In order to achieve an understanding of the impact of various practical methods of concrete placement, and to recommend favorable methods, production trials and shear testing need to be undertaken.

The following factors are considered critical in the testing program. Proposed methods of addressing each factor are presented.

1. **Control specimens.** We need to start with a typical design with average parameters. Testing three specimens with these conditions would allow for better understanding of consistency and repeatability of results, and would allow for comparison with prediction under average conditions. In producing these specimens, the concrete is placed progressively in several runs from one end to the other until the forms are filled (3 specimens).

2. **Fiber contribution.** Test one specimen without fibers but with all other parameters kept the same as in Item 1. Test a second specimen with the shorter 12 mm fibers. (2 specimens)

3. **Level of prestress.** Check the impact of prestress level on $\beta$ on $\theta$. One specimen with no prestress, one with 50% of the prestress in Item 1. (2 specimens)

4. **Concrete placement.** Use two methods: one with the concrete only placed at mid-length and the second with concrete progressively placed along the length from one end to the other in several runs. (1 specimen)

5. **Member shape and size.** The basic form is the 2 ft 10 in. deep I-beam. It is used unless indicated otherwise. The same design used in Item 1, is repeated here with the 4 ft-6 in. Deep I beam (1 specimen), with voided slab (1 specimen) and with the decked I-beam (1 specimen), and the bridge box beam (1 specimen). Also, note that the specimens include both radiused and chamfered transitions between the web and flange, and the ratio of flange area to web area varies. As such, under the proposed testing program, the effect of these different details on behavior and shear strength can be assessed.

6. **Tension tie anchorage.** All specimens will be designed and detailed to satisfy this requirement. Two different methods are used. The first one is to allow the beam to overhang enough length to satisfy the tension tie requirement with the provided flexural reinforcement. The second method is to provide supplementary reinforcement when no space is available to adequately overhang the member. This will be repeated with two specimens, an I-beam and a voided slab. In these two cases, extra steel bars and/or plates will be included in the detailing of the member end (2 specimens).

7. **Shear span.** All of the tests are done with a shear span to total depth ratio of about 2.5. One test of an I-beam will be done with a ratio of 3.5. (1 specimen) to assess the impact on crack angle and overall behavior.

8. **Stirrups.** Add stirrups to the design in Item 1 and test. Use a medium level of stirrup and a high level of stirrup reinforcement. It is possible that the contributions of concrete, fibers and stirrups to shear resistance will drive the capacity to a level near the maximum allowed for control of compression strut failure. Testing will allow for observation of compression strut behavior (2 specimens).

9. **Web width.** Repeat testing in Item 1 with 2 in. and again with 4 in. webs (2 specimens)

Total number of specimens: 19.

**Figure 7.8-1** shows the proposed shear test specimens. **Table 7.8-1** summarizes some details of the specimens showing the predicted shear failure load.
Figure 7.8-1. Proposed Shear Test Specimens
### Table 7.8.1. Summary of the Proposed Shear Test Specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Designation*</th>
<th>Parameter</th>
<th>Beam Type</th>
<th>Fiber Type</th>
<th>Shear Span, a (in.)</th>
<th>Beam Height, h (in.)</th>
<th>a/h</th>
<th>Web Width, b, (in.)</th>
<th>Prestress</th>
<th>Flexural Rebar</th>
<th>Stirrups</th>
<th>Overhang** (in.)</th>
<th>Total Length (ft)</th>
<th>Predicted Ultimate Load (kip)</th>
<th>Flexure/Shear Capacity</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A3aS0P2-1</td>
<td>Baseline</td>
<td>A</td>
<td>a</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>3</td>
<td>26-0.6*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>18</td>
<td>700</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>A3aS0P2-2</td>
<td>Baseline</td>
<td>A</td>
<td>a</td>
<td>84</td>
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<td>0</td>
<td>0</td>
<td>24</td>
<td>18</td>
<td>700</td>
<td>1.5</td>
<td></td>
</tr>
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<td>A3aS0P2-3</td>
<td>Baseline</td>
<td>A</td>
<td>a</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>3</td>
<td>26-0.6*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>18</td>
<td>700</td>
<td>1.5</td>
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<tr>
<td>4</td>
<td>A30S3P2</td>
<td>Fibers replaced with stirrups</td>
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<td>84</td>
<td>34</td>
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<td>3</td>
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<td>#4@4&quot;</td>
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<td>18</td>
<td>690</td>
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<td></td>
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<td>5</td>
<td>A36bS0P2</td>
<td>Short fiber</td>
<td>A</td>
<td>b</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>3</td>
<td>26-0.6*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>18</td>
<td>700</td>
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<tr>
<td>6</td>
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<td>50% prestress</td>
<td>A</td>
<td>a</td>
<td>84</td>
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<td>a</td>
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<td>8</td>
<td>A3aS0P2-C</td>
<td>Favorable concrete placement for fiber orientation</td>
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<td>a</td>
<td>84</td>
<td>34</td>
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<td>24</td>
<td>18</td>
<td>700</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>B3aS0P3</td>
<td>Larger I beam depth</td>
<td>B</td>
<td>a</td>
<td>135</td>
<td>54</td>
<td>2.50</td>
<td>3</td>
<td>26-0.7*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>26.5</td>
<td>1120</td>
<td>1.3</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>DB4aS0P2</td>
<td>Bridge decked I beam</td>
<td>DB</td>
<td>a</td>
<td>98.4</td>
<td>39.4</td>
<td>2.50</td>
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<td>14-0.6*</td>
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<tr>
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<td>BS6aS0P2</td>
<td>Bridge box slab</td>
<td>BS</td>
<td>a</td>
<td>45</td>
<td>18</td>
<td>2.50</td>
<td>3 plus 3</td>
<td>22-0.5*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>11.5</td>
<td>TBD</td>
<td>TBD</td>
<td>To be tested at Windsor Univ. or NCSU</td>
</tr>
<tr>
<td>12</td>
<td>V54bS0P2</td>
<td>Building voided slab</td>
<td>VS</td>
<td>b</td>
<td>55</td>
<td>22</td>
<td>2.50</td>
<td>2 plus 2</td>
<td>16-0.5*</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>9.67</td>
<td>TBD</td>
<td>TBD</td>
<td>End plate and bar to enhance anchorage</td>
</tr>
<tr>
<td>13</td>
<td>A3aS0P2-S</td>
<td>Short end embedment with supplementary rebar</td>
<td>A</td>
<td>a</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>3</td>
<td>26-0.6*</td>
<td>0</td>
<td>0</td>
<td>12</td>
<td>16.0</td>
<td>700</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>V54bS0P2-S</td>
<td>Short end embedment without supplementary rebar</td>
<td>VS</td>
<td>b</td>
<td>55</td>
<td>22</td>
<td>2.50</td>
<td>2 plus 2</td>
<td>16-0.5*</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>9.67</td>
<td>TBD</td>
<td>TBD</td>
<td>No anchorage enhancement</td>
</tr>
<tr>
<td>15</td>
<td>A3aS0P2-L</td>
<td>Long shear span</td>
<td>A</td>
<td>a</td>
<td>119</td>
<td>34</td>
<td>3.50</td>
<td>3</td>
<td>26-0.6*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>23.8</td>
<td>700</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>A3aS1P2</td>
<td>Supplementary stirrups, medium</td>
<td>A</td>
<td>a</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>3</td>
<td>26-0.6*</td>
<td>0</td>
<td>#3@12&quot;</td>
<td>24</td>
<td>18.0</td>
<td>788</td>
<td>1.4</td>
<td>Maximum allowable shear may be exceeded</td>
</tr>
<tr>
<td>17</td>
<td>A3aS2P2</td>
<td>Supplementary stirrups, heavy</td>
<td>A</td>
<td>a</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>3</td>
<td>26-0.6*</td>
<td>0</td>
<td>#4@12&quot;</td>
<td>24</td>
<td>18.0</td>
<td>860</td>
<td>1.2</td>
<td>Maximum allowable shear may be exceeded</td>
</tr>
<tr>
<td>18</td>
<td>A2aS0P2</td>
<td>Narrow I beam web</td>
<td>A</td>
<td>a</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>2</td>
<td>26-0.6*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>18.0</td>
<td>491</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>A4aS0P2</td>
<td>Wide I beam web</td>
<td>A</td>
<td>a</td>
<td>84</td>
<td>34</td>
<td>2.47</td>
<td>4</td>
<td>26-0.6*</td>
<td>0</td>
<td>0</td>
<td>24</td>
<td>18.0</td>
<td>900</td>
<td>1.2</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
- All mild reinforcement is ASTM A1035, Grade 100 ksi.
- All members reinforced with bursting vertical steel = 0.04P/,20 ksi.
- Except for Specimen A3aS0P2-C, all specimens are produced with concrete placed progressively from one end to the other end in multiple rounds.
- A3aS0P2L-1 Cross section shape: Type A = I beam, 34" deep; Type B = I beam, 54" deep; Type DB = decked I beam; Type BS is bridge box slab; Type VS is building voided slab.
- Web width in inches.
- Fiber Type: a = 20 mm; b = 12 mm; 0 = no fibers.
- Stirrups: 0 = no stirrups, 1 = #3@12", 2 = #4@12"; 3 = #4@4".
- Prestress level: 0 = no prestress; 1 = 12-0.6" Stressed at 75% of GUTS; 2 = 26-0.6" stressed at 75% of GUTS; 3 = 26-0.7" stressed at 65% of GUTS.
- L, C, S Special feature: L = long shear span/depth ratio; C = Concrete placed at mid length with tremie; S = short end anchorage.
- 1, 2, 3 Specimen number of a group of identical specimens.

**Overhang length from specimen end to centerline of bearing device.**
Vertical Shear Design

The expected test set-up is illustrated in Figure 7.8-2. To reasonably assure sectional shear behavior, as opposed to strut action, the shear span is generally about 2.5 times the specimen depth or more. Based on prior research, a target design strength and corresponding service load will be estimated for all specimens. The test load will be applied in at least four equal increments to the service load, which will be held for at least 30 minutes. Cracking, strain and deflection will be evaluated and at each load level and also during the 30-minute hold period. The specimens will then be loaded in equal increments to failure. Again, cracking, strain and deflection will be recorded at each load level.

As illustrated in Figure 7.8-2, the test load will be monitored with a calibrated load cell. Deflection at the center and each end will be monitored using string potentiometers. Diagonal strain will be measured at mid depth, midway between the load and support, using pi gages. Pi gauges are displacement transducers mounted to the concrete surface that measure strain over a fixed length, typically 8 in. These measurements will provide a rational calculation basis for shear cracking angle throughout the load test. Linear variable differential transformers (LVDTs) will be attached to extended strands/rebars from each end of the beam to determine if there is any reinforcement slippage. The LVDTs may be eliminated if the research team decides to bend the rebars inside the beam at each end. Strain gages will be provided at the bottom of the beam to measure the strain at the centroid of longitudinal reinforcement at the location of applied force.
Implementation of UHPC in Long-Span Precast Pretensioned Elements

**Figure 7.8-2: Vertical Shear Test Setup**

- **a) Elevation View of Beam Shear Test Setup**
- **b) Section View of Beam Shear Test Setup**
7.9 Pilot Testing: Building Voided Slab

Two shear tests were performed on the 60-ft voided slab specimen used for flexural tests as reported earlier in Section 5.8. The purpose of these tests was to determine the ultimate shear capacity of the voided slab. Each end was tested as shown in Figure 7.9-1. The embedment length was 7 in. from the end of the specimen to the internal face of the support nearby the shear span. On end only was provided with end plate to improve the anchorage of the strands. String potentiometers measured vertical deflection of the stems underneath the load point closest to the primary support. Linear potentiometers were used to measure the vertical deflection of the support itself so that any support motion could be subtracted from other measurements.

Each end of the specimen was loaded individually to selected support reactions calculated from the self-weight and a 60' by 6' tributary area. A 100 psf live load and a MEP dead load allowance of 15 psf were assumed. The specimen was loaded and briefly held at the following levels: simulated dead load, full-service load, full factored load, and additional load increments between factored level and failure. The load was held at each level for the brief period of time necessary to examine the specimen and document/photograph any observations (typically less than 5 minutes at each level). Loads were applied to the specimen at the increments designated in Table 7.9-1.

**Table 7.9-1. Loading Sequence Used in Both Shear Tests**

<table>
<thead>
<tr>
<th>Load Step</th>
<th>Description</th>
<th>Stem Reaction (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dead Load</td>
<td>15.2</td>
</tr>
<tr>
<td>2</td>
<td>Service Load (DL + LL)</td>
<td>33.2</td>
</tr>
<tr>
<td>3</td>
<td>Factored Load (1.2<em>DL + 1.6</em>LL)</td>
<td>47.1</td>
</tr>
<tr>
<td>4</td>
<td>Load level</td>
<td>80</td>
</tr>
<tr>
<td>5</td>
<td>Load level</td>
<td>100</td>
</tr>
<tr>
<td>6</td>
<td>Up to failure</td>
<td>increased as necessary</td>
</tr>
</tbody>
</table>

The end with anchorage improvement and bursting reinforcement was tested first. Cracking was first observed on the right stem at a support reaction of 80 kips (Figure 7.9-2). The initial crack was mainly longitudinal near the top flange of the specimen with a diagonal crack developing at the end closest to the support. This crack continued
to propagate until the ultimate failure occurred at a support reaction of 109 kips (Figure 7.9-3). The failure appeared to be controlled by horizontal shear along this longitudinal crack, which propagated all the way to the end of the specimen (Figure 7.9-4). The diagonal crack terminated at the bottom flange (Figure 7.9-4).

Video of the failure indicates that the longitudinal crack extending to the end of the specimen allowed a local flexure failure to take place in the web. This local failure developed at nearly the same instant the longitudinal crack extended to the end of the specimen. At this instant, a section of the specimen appears to have rotated in flexure with a vertical crack developing near the support and a vertical line of crushing developing near the applied load. It appears the vertical crack near the support developed at a steel reinforcing bar (Figure 7.9-5). No strand slip was observed, even after failure (strands were marked before testing and checked after failure).

It is important to note that only the right stem failed. The left stem remained un-cracked during the test. The right stem was observed to be significantly thinner at the end of the specimen (1-5/8”), as compared to the left stem at the same location (2-5/8”).

Figure 7.9-2. Cracking in Web of Right Stem at Support Reaction of 80 kips

Figure 7.9-3. Crack Pattern in Web of Right Stem After Failure (Support Reaction of 109 kips)
The shear test setup was installed on the opposite end of the beam and loads were applied to the second end in the same increments listed in Table 7.9-1. The specimen remained un-cracked through a support reaction of 100 kips. At a support reaction of 108 kips, the topping slab abruptly slid and lifted off at the back-transverse joint. This failure coincided with the failure of vertical grout pockets nearest to this joint. Loading was continued through this event until ultimate failure of the specimen was achieved at a support reaction of 117 kips (Figures 7.9-6 and 7.9-7). Both stems failed simultaneously in shear with large diagonal cracks being the obvious failure mode. On this end, the webs of both stems were measured to be 2” thick (as detailed), prior to the experiment beginning. Failure spanned the entire width of the beam and encompassed the bottom slab. Strand slip of approximately 3/8” was measured in the (12) bottom prestressing strands (6 strands per stem) after failure. Load-deflection curves for shear tests at both ends can be seen in Figure 7.9-8. Note that the theoretical capacity seems to match well the experimental results and that both capacities are significantly larger than the demand (load).
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Vertical Shear Design

Figure 7.9-6. Right Stem After Failure (Support Reaction of 117 kips)

Figure 7.9-7. Left Stem After Failure (Support Reaction of 117 kips)

Figure 7.9-8. Load-Deflection Curves for Testing With and Without Additional Tension Tie Anchorage

Test with additional tension tie anchorage
Test with no additional tension tie anchorage
Required shear resistance
Nominal shear resistance
7.10 References

ACI Committee 318. (2014). *Building Code Requirements for Structural Concrete and Commentary*, (ACI 318-14), American Concrete Institute: Farmington Hills, MI, p. 503.


German Committee for Structural Concrete (Deutscher Ausschuss für Stahlbeton) (DAfStb) (2017). *Ultra-High-Performance Concrete*, Berlin, Germany.


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8 INTERFACE SHEAR BEHAVIOR

8.1 Notation

\[ A_{cv} = \text{area of concrete considered to be engaged in interface shear transfer (in.}^2) \]

\[ A_{vf} = \text{area of interface shear reinforcement crossing the shear plane within the area } A_{cv} \text{ (in.}^2) \]

\[ B = \text{flute Length (in.)} \]

\[ c = \text{cohesion factor (ksi)} \]

\[ f_v = \text{specified minimum yield strength of reinforcement (ksi)} \]

\[ P_c = \text{permanent net compressive force, normal to the shear plane (kip)} \]

\[ V_{ni} = \text{nominal interface shear resistance (kip)} \]

\[ v_{ni} = \text{nominal interface shear strength (ksi)} \]

\[ \mu = \text{friction factor} \]

\[ \rho_v = \text{ratio of area of vertical shear reinforcement to area of gross concrete area of a horizontal section} \]

8.2 Introduction

The ability to transfer shear across an interface is an important factor for the performance of bridges and buildings that rely on field toppings and/or wet joints. The interface shear capacity across a cold-joint depends on the configuration of that interface plane. There are cases where UHPC is used on both sides of the interface. When the interface is fluted (corrugated) in such a way that the fibers are engaged in the potential shear failure plane, then the effect of fibers should be accounted for.

8.3 Previous Work and Current International Practices

The hypothesis of shear behavior of a concrete to concrete interface was presented by Birkeland and Birkeland (1966). The horizontal shear resistance was attributed to the friction resulting from either the external load, as shown in Figure 8.3-1a, or the clamping force from reinforcement crossing the interface plane, as shown in Figure 8.3-1b. If the roughening of the surface is considered as saw teeth, it may be concluded that the horizontal resistance force is a function of the angle of the teeth, \( \bar{\theta} \). The horizontal component could be calculated from the clamping force times \( \tan \bar{\theta} \). The \( \tan \bar{\theta} \) function is called the friction coefficient, \( \mu \). This work was part of connections of precast construction, so there was no introduction of the cohesion effect for wet joints.

![Figure 8.3-1. Shear Friction Hypothesis](image)

Mattock and Hawkins (1972) introduced the effect of cohesion for initially cracked specimens to determine the horizontal shear strength as shown in Eq. 8.3-1. It was proposed that the cohesion factor, \( c \), and friction factor, \( \mu \), are 0.20 ksi and 0.8, respectively. They performed push-off tests on specimens with a maximum compressive strength of 4.0 ksi. A limited number of specimens with compressive strengths of 6.0 ksi were also tested. The interface shear strength was limited to \( 0.3 f'_c \).
\[ V_{ni} = cA_{cv} + \mu (A_{vf} f_v + P_c) \leq 0.3f'_c A_{cv} \]

AASTHO LRFD has adopted the equation proposed by Mattock and Hawkins (1972) with different values of \( c \) and \( \mu \) to account for the configuration of the interface plane. For simplicity, the “cohesion factor”, \( c \), is used in AASHTO LRFD to capture the effects of cohesion and/or aggregate interlock. However, cohesion provided by cementitious bond and the shear-friction contribution of reinforcement cannot co-exist because the latter requires separation across the interface to develop strain in the shear-friction reinforcement. Thus, when friction from a normal force provided by reinforcement is combined with the “cohesion” contribution, the latter is actually the contribution of aggregate interlock or other effects related to the concrete area.

The ultimate nominal strength limit also varies depending on the interface surface, with a maximum value of 1.80 ksi, which controls when \( f'_c \) is more than 6 ksi. It appears that this upper limit is too conservative when using UHPC that has a compressive strength of 18 ksi or higher. AASHTO LRFD specifies that this equation should be applied to the interface plane with a minimum amount of crossing reinforcement providing at least 0.050 ksi of clamping stress. The amount of interface shear reinforcement should still not exceed 1.33 of the factored interface shear force, \( V_{ni} \), divided by the reduction factor, \( \phi \). On the other hand, this minimum reinforcement may be waived in the case of a roughened surface with a minimum amplitude of 1/4 in. and a factored interface shear stress less than 0.210 ksi.

ACI 318 also specifies similar equations to AASHTO LRFD, but with lower values of \( c \) and \( \mu \) for the same surface conditions with a minimum crossing reinforcement requirement. ACI 318 substituted the shear strength limit of 0.3\( f'_c \) with 0.50 ksi. If the ultimate factored shear stress is higher than this limit, the cohesion effect shall be neglected (the first term in the AASHTO LRFD equation). If the shear stress at the interface exceeds 0.5 ksi, ACI 318 equations do not consider cohesion. If the shear stress is less than 0.5 ksi, the ACI 318 resistance equation includes both a cohesion and shear-friction term. The minimum amount of interface shear reinforcement must provide a clamping stress of 0.024\( \sqrt{f'_c} \), but not less than 0.050 ksi. Therefore, the demand for reinforcement crossing the interface plane is greater when the ACI calculation model is followed.

Crane (2010), Maroliya (2012), and Jang et al. (2017) studied the interface shear resistance of Monolithic UHPC. Carne (2010) performed push-off tests on UHPC specimens with a fiber content of 2% by volume and an interface area of 87 in.\(^2\). One group of tests was done using no reinforcement, and one group of tests was done using a reinforcement level equal to 0.5% of the interface area. It was reported that the cohesion factor and friction factor are 1.9 ksi and 4.1, and 2.6 ksi and 4.5 for the pre-cracked and uncracked planes, respectively. These values are much higher than the 0.4 ksi and 1.4 values specified by AASHTO for conventional concrete. Maroliya (2012) tested inverted L-shaped UHPC specimens by applying direct loading on the interface plane between the flange and the web. Three interface shear areas of 13.9, 16.3, and 18.6 in.\(^2\) were tested. For UHPC with a fiber content of 2% by volume, the cohesion factor was reported as 1.8 and 2.2 ksi for moist curing and heat curing, respectively. Jang et al. (2017) conducted push-off tests without crossing reinforcement. The cohesion factor was reported as 2.73 ksi for UHPC with a fiber content of 1.5% by volume and an interface shear plane area of 46.6 in.\(^2\).

Harris et al. (2011), Tayeh et al. (2012), Muñoz (2012), Rangaraju et al. (2014), and Aaleti and Sritharan (2017) evaluated the interface shear resistance between UHPC cast against hardened conventional concrete with a slant shear test (ASTM C882). In these tests, cylinders or prisms were cast at two stages with a slanted interface plane that was inclined at angle \( \theta \) from the horizontal axis. The interface surfaces of the conventional concrete (first cast) were prepared with different roughened surfaces such as sandblasted, brushed, or grooved (fluted). The composite specimens were tested under compression loading with two components parallel and perpendicular to the slanted plane. The perpendicular component, \( P \cos \theta \), represents the clamping force from either the reinforcement or an external force, while the parallel component, \( P \sin \theta \), represents the interface shear strength. The two components of all the test results were divided by the interface area and plotted as shown in Figure 8.3-2. The slope of the best fit line is considered the friction factor. The cohesion factor can be determined by the intersected part of the vertical axis. The data collected from the literature was divided into three categories according to the surface texture; sandblasted, brushed, and grooved. The friction and cohesion factors for different roughened surfaces are summarized in Table 8.3-1. This table also presents the factors for conventional concrete, as specified by AASHTO.
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Interface Shear Behavior

**Figure 8.3-2. Relationship Between Interface Shear Stress and Normal Stress of CC-UHPC**

**Table 8.3-1. Summary of the Interface Shear Resistance of CC-UHPC**

<table>
<thead>
<tr>
<th>Surface Texture</th>
<th>c (ksi)</th>
<th>μ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandblasted</td>
<td>0.52</td>
<td>1.45</td>
</tr>
<tr>
<td>Brushed</td>
<td>0.52</td>
<td>1.12</td>
</tr>
<tr>
<td>Grooved (fluted)</td>
<td>0.80</td>
<td>1.04</td>
</tr>
<tr>
<td>Roughened (AASHTO LRFD)</td>
<td>0.24</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Banta (2005) and Crane (2010) also evaluated the interface shear resistance between lightweight and high-strength conventional concrete, respectively, cast against hardened UHPC via push-off tests. Banta (2005) studied four different surface preparations: deformed, chipped, fluted, and smooth surface. The specimens with a smooth surface had different reinforcement ratios to provide clamping stresses between 0-0.26 ksi. Carne (2010) also studied three different surface preparations: burlap-roughened, fluted, and smooth surface. The specimens with a smooth surface had different reinforcement ratios providing clamping stresses between 0-0.45 ksi. The test results from Banta (2005) and Crane (2010) were analyzed and plotted as shown in **Figure 8.3-3**. It shows the relation between the interface shear stresses and clamping stresses of the smooth surface between UHPC and CC. It shows the cohesion and friction factors as 0.14 ksi and 1.05, respectively. These values are higher than the AASHTO LRFD provisions of 0.075 ksi and 0.6 for cohesion and friction factors, respectively. Crane (2010) also reported that the preparation of the interface plane significantly affected the interface shear strength. It was reported that roughening with burlap increased the interface shear strength of a cold joint by 127% without adding any reinforcement. The fluted surface increased the strength by 228% with no reinforcement. Additionally, providing clamping stresses of 0.30 ksi could enhance the behavior of the fluted joint by 120%.
Jang et al. (2017) utilized push-off testing to evaluate the interface shear resistance between UHPC cast on hardened UHPC and on conventional concrete. They studied five different surface preparations: water jet roughened, 0.4” fluted, 0.8” fluted, 1.2” fluted, and smooth surface. There was no reinforcement crossing the interface plane. **Figure 8.3-3** presents the effect of the surface texture on the interface shear strength for UHPC against UHPC and UHPC against CC. The difference between water jet and smooth is due to an improvement in cohesion due to cementitious bond. On the other hand, the very high resistance of the 0.8-in. and 1.2-in. fluted samples is due to forced shearing of individual teeth. The UHPC-UHPC performed better because the teeth were stronger.

**Figure 8.3-3. Relation Between Interface Shear Stress and Clamping Stress of a Smooth Surface UHPC-CC**

**Figure 8.3-4. Relation Between Interface Shear Stress and Surface Texture of the Interface Shear Plane**
Liu et al. (2018) have recently reported on the results of a comprehensive testing program to investigate interface shear capacity of fluted (shear keyed) dry connections of segmental box girder bridges. This is the first comprehensive study on that topic. Two key sizes were considered: 50 by 300 mm (2-in. by 12-in.) and 50 by 100 mm (2-in. by 4-in.). Specimens without any reinforcing bars as well as specimens with special reinforcement were tested. The results showed a shearing capacity in the order of 12 to 24 MPa (1.74 to 3.48 ksi), which is a very high capacity and is consistent with previous work on fluted UHPC-UCP interface, with one of the components being precast and the other being CIP placed against the first one. This information is very encouraging. It opens the door for more creative precast concrete UHPC systems made in parts and connected with dry connections. Obviously, a connection with the aid of a joint compound or epoxy would ensure a tight fit and produce similar results. The only application known to the research team at this time is the segmental UHPC bridge work in Malaysia by Dr. Voo, Phase II of this study will take advantage of the information in Liu et al. and further develop an AASHTO type interface design formulation for it.

While there is limited research done to evaluate the interface shear between full-scale-UHPC beams and cast-in-place CC topping, Carne (2010) tested six I-Beams topped with conventional concrete. The beams have a depth of 32.4" with different top flange widths. Three specimens have a smooth interface surface with clamping stresses of 0, 0.062, and 0.250 ksi. The other three have a ¼"-fluted surface with clamping stresses of 0, 0.062, and 0.250 ksi. The clamping stresses were provided from transverse reinforcement with 12 or 24 in. spacing. The relative displacement between the top of the beam and the deck was monitored during the flexural test. The interface shear strength was determined based on changes in either the initial slope of shear-relative displacement or the load-deflection curve, rather than using a certain value of the relative displacement. For the smooth surface interface without reinforcement, no composite action was reported. The interface shear strength of the beam with a smooth interface was overestimated when Eq. 8.3-1 was utilized with clamping stress, cohesion factor, and friction factor of 0.062 ksi, 0.075 ksi, and 0.6, respectively. On the other hand, the same equation underestimated the strength of the beams with a fluted surface, with or without reinforcement. This is because the very high resistance of the fluted samples is due to the forced shearing of individual teeth.

AFGC (2013) has adapted Eurocode 2 provisions with some modifications to predict the interface shear strength of UHPC. Unlike AASHTO LRFD, Eurocode 2 proposed the cohesion term to be a function of the cohesion factor and the tensile properties of the conventional concrete. The Eurocode specifies dimensions for a fluted construction joint, which is also adapted by AFGC (2013) as shown in Figure 8.3-5. Any dimensions of the fluted joint that are parallel to the interface plane, such as \( b \), \( h_1 \), and \( h_2 \), should not be less than double the length of the fiber with a minimum depth of half the length of the fibers. The depth should not be less than 0.2". The dimensions in the fluted direction, such as \( h_1 \) and \( h_2 \), should not exceed 10 times the depth. For fluted joints, an additional term was added to introduce the strength provided by the fibers crossing the interface plane, as shown in Eq. 8.3-2. This term is only applicable for fluted joints. Therefore, minimum dimensions of the fluted joint are proposed to be a function of the fiber length. While the cohesion term is a function of the first crack tensile strength, the fiber term is a function of the post-cracking tensile strength. AFGC (2013) limits the maximum nominal interface shear strength to \( 0.5f_{c}^{1/2}\) for UHPC.

\[
V_{ni} = c_{f_{el}A_{cv}} + A_{of}f_{y} + \mu_{c} + (0.35\mu + 0.3)f_{c,f}/K \leq 0.5f_{c}^{1/2}A_{cv}
\]

Eq. 8.3-2

where

- \( f_{c,el} \) = limit of elasticity under tension obtained on prism (ksi)
- \( f_{c,f} \) = maximal post-cracking stress (ksi)

![Figure 8.3-5. Fluted Joint Details as Specified by AFGC (2013)](image-url)
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**Interface Shear Behavior**

\[
K = \text{fiber orientation factor}
\]

\[
c = \text{cohesion factor}
\]

\[
\mu = \text{friction factor}
\]

The following values shall be taken for cohesion factor, \(c\), and friction factor, \(\mu\):

- For UHPC placed against a clean UHPC surface, which is previously formed:
  \[
  c = 0.025 - 0.10
  \]
  \[
  \mu = 0.5
  \]

- For UHPC placed against a clean UHPC surface, which is previously unformed:
  \[
  c = 0.2
  \]
  \[
  \mu = 0.6
  \]

- For UHPC placed against a clean UHPC surface, which is roughened using a formliner:
  \[
  c = 0.4
  \]
  \[
  \mu = 0.7
  \]

- For UHPC placed against a clean UHPC surface that is fluted:
  \[
  c = 0.5
  \]
  \[
  \mu = 1.4
  \]

### 8.4 Proposed Tentative Design Recommendations

It may be allowed to have a smooth interface surface between two different casts of concrete. The interface plane may also be fluted or roughened with a minimum amplitude of \(\frac{1}{4}\)". The roughened surface should be performed using a form liner. **Figure 8.4-1** shows the proposed fluted surface of UHPC joints with a flute length, \(b\), of 6 in. to 12 in. for fiber lengths of \(\frac{1}{2}\)" and \(\frac{3}{4}\)". Eq. 8.4-1 presents the proposed equation to predict the nominal interface shear strength of UHPC. The proposed values for cohesion and friction factors are shown below, and are subject to be revised by the testing program of Phase II.

\[
V_{ni} = cA_{cv} + \mu(A_{ef} f_y + P_c) \leq 0.3f'_c A_{cv}
\]

**Equation 8.4-1**

The following values are proposed for cohesion factor, \(c\), and friction factor, \(\mu\):

- For UHPC placed against a clean UHPC surface that is fluted:
  \[
  c = 1.0 \text{ ksi}
  \]
  \[
  \mu = 1.0
  \]

- For CC placed against a clean UHPC surface that is fluted or roughened:
  \[
  c = 0.4 \text{ ksi}
  \]
  \[
  \mu = 1.4
  \]

- For CC placed against a clean UHPC surface, free of laitance, but not intentionally roughened:
  \[
  c = 0.075 \text{ ksi}
  \]
  \[
  \mu = 0.6
  \]
8.5 Example for Fiber Contribution to Interface Shear Strength

The bridge construction has high demand for interface shear due to the required composite action between the beams and the deck. This may be more critical for sections that have a narrow top flange, creating a limited interface surface. UHPC Tubs with only 12-in.-wide top flanges are a good example for presenting the required interface shear reinforcement by different codes. It is proposed that a fluted surface is created at the top flange of the tub. This girder spans 235 feet, is simply supported, and is topped with a precast concrete deck. The joint and haunch should be poured with UHPC. The interface area is 2 in. x 8 in. = 16 in. The interface plane is fluted as shown in the preceding section. Figure 8.5-1 shows the cross section of the tub girder bridge and the connection between the deck and the tub.

Figure 8.5-1. Cross-Section of the Tub Girder Bridge

Figure 8.5-2 shows the required interface shear reinforcement in accordance with ACI 318-14, AASHTO LRFD 8th, AFGC (2013), and the PCI proposed approach. ACI 318-14 and AASHTO LRFD 8th account for minimum reinforcement requirements for normal weight concrete placed against a clean concrete surface, free of laitance, with the surface intentionally roughened to an amplitude of 0.25 in. While ACI 318 shows a higher demand of reinforcement than AASHTO, AFGC (2013) and the PCI proposed approach show no need for interface shear reinforcement as long as a fluted interface plane with a minimum amplitude of 0.5 in. is used.
8.6 Plans for Phase II Testing

Interface shear testing will be conducted at two levels: 1) push-off specimens; and 2) full-scale beam specimens. Based on the results of the push-off test, the most promising connections will be further developed and tested in a full-scale beam test configuration.

8.6.1 Push-Off Specimens

It is envisioned that specialized push-off component testing will provide conservative characterization of UHPC interface shear. Push-off testing is one of the most common and practical methods for interface shear evaluation. It has been shown by many researchers to be more conservative and much less expensive than full composite beam tests. The objective of the proposed push-off testing is to determine the interface shear resistance for different interface conditions. Both interface conditions of UHPC against UHPC and UHPC against conventional concrete will be included. The interface will be tested for different surface conditions. Figure 8.6-1 shows the proposed push-off shear specimens’ configuration, which is similar to that used by Mattock and Hawkins (1972). The interface plane includes monolithic casting, smooth interface, and fluted (formed) interface. There seems to be difficulty in using conventional raking or brushing methods with UHPC to create the ¼ in. roughening required by codes. Therefore, in this research, we will experiment with methods of creating corrugations that will provide for positive mechanical interlock between the UHPC surface and the cast-in-place topping. Three main categories are investigated: hardened UHPC against hardened UHPC, UHPC cast against UHPC, and conventional concrete cast against UHPC.
Interface Shear Behavior

Table 8.6-1 lists the combinations planned to be tested. All specimens have an interface plane of 5 in. x 12 in. Specimen #1 is monolithically cast with no reinforcement crossing the interface plane, and is used as a reference for comparison. Specimens #2 through #4 represent the application of a fresh conventional concrete cast slab on a hardened UHPC slab/girder using two different preparations for the interface surface. High strength steel, ASTM A1035, and mild steel, ASTM A615 Gr. 60, will be utilized for interface reinforcing. Specimen #5 represents the application of using fresh UHPC to fill the haunch or joint between precast UHPC components with a fluted interface surface. As shown in the UHPC product development section, it is envisioned to produce some of the products in two parts and then connected in the yard/field using embedded 3 in. post-tensioning sleeves filled with UHPC. Therefore, specimens #6 through #12 represent the situation of two precast UHPC pieces. Those specimens have a fluted surface. Specimens #6 and #7 evaluate the effect of not applying any adhesive before filling the sleeves with UHPC. Specimens #8 and #9 evaluate the effect of a compound adhesive being applied before filling the sleeves with UHPC. Specimens #10 through #12 evaluate the effect of using an epoxy adhesive that is applied before filling the sleeves with UHPC. The difference between specimens #10, #11, and #12 is the effect of monolithic UHPC crossing the interface plane. It should be noted that the specimens in this research were developed to include the most probable products for implementation or as a base line for future expansion. All the specimens will be cast and tested in the UNL Laboratory.

**Figure 8.6-1. Push-Off Test Specimens**
Table 8.6-1. Test Matrix for Push-off Specimens

<table>
<thead>
<tr>
<th>Test</th>
<th>No. of Specimens</th>
<th>First Cast</th>
<th>Second Cast</th>
<th>UHPC Surface Type</th>
<th>Reinf.</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>UHPC</td>
<td>None</td>
<td>Monolithic</td>
<td>None</td>
<td>Reference</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>UHPC</td>
<td>Fresh CC</td>
<td>Smooth</td>
<td>¾” HS rod</td>
<td>CIP CC Deck Against Precast UHPC Beams</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>UHPC</td>
<td>Fresh CC</td>
<td>Fluted</td>
<td>¾” HS rod</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>UHPC</td>
<td>Fresh CC</td>
<td>Fluted</td>
<td>2#4 Gr. 60</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>2</td>
<td>UHPC</td>
<td>Fresh UHPC</td>
<td>Fluted</td>
<td>¾” HS rod</td>
<td>CIP UHPC Closure Pour Against Precast UHPC Components</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>UHPC</td>
<td>Two Hardened UHPC Components with Sleeve and Dry Joint</td>
<td>Fluted</td>
<td>¾” HS rod</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>UHPC</td>
<td>Two Hardened UHPC with Sleeve and Dry Joint</td>
<td>Fluted</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>UHPC</td>
<td>Two Hardened UHPC with Sleeve and Compound</td>
<td>Fluted</td>
<td>¾” HS rod</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>UHPC</td>
<td>Two Hardened UHPC with Sleeve and Compound</td>
<td>Fluted</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>2</td>
<td>UHPC</td>
<td>Two Hardened UHPC with Sleeve and Epoxy</td>
<td>Fluted</td>
<td>¾” HS rod</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>2</td>
<td>UHPC</td>
<td>Two Hardened UHPC with Sleeve and Epoxy</td>
<td>Fluted</td>
<td>None</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td>UHPC</td>
<td>Two Hardened UHPC with Epoxy only</td>
<td>Fluted</td>
<td>None</td>
<td></td>
</tr>
</tbody>
</table>

Test results will be compared with a predicted nominal shear resistance that is calculated using Eq. 8.4-1. The critical shear plane may not be obvious by initial inspection. All plausible crack lines should be investigated, as shown in Figure 8.6-3. For each configuration, the section that produces the smallest capacity will become the
limiting value. If a crack line passes through two different interface configurations, the term \((c_1A_{cv1} + \mu_1A_{vf1}f_y)\) may be repeated with different subscripts, such as \((c_2A_{cv2} + \mu_2A_{vf2}f_y)\). A third term \((c_3A_{cv, sleeves} + \mu_3A_{vf3}f_y)\) represents the contribution of a grouted sleeve that passes through the interface where two precast components are attached to each other (e.g., dry joint). This will result in Eq. 8.6-1. If the crack line passes through a sleeve that is both embedded in precast UHPC and also filled with UHPC, the third term may be included within the first term. For surfaces other than fluted ones, only one crack line should be considered.

**Figure 8.6-3. Fluted Connection Between Two Hardened UHPC Parts**

\[ V_{ni} = (c_1A_{cv1} + \mu_1A_{vf1}f_y) + (c_2A_{cv2} + \mu_2A_{vf2}f_y) + (c_3A_{cv, sleeves} + \mu_3A_{vf3}f_y) \]

Eq. 8.6-1

Table 8.6-2 lists tentative friction and cohesion factors for different cases. These values are summarized from AASHTO LRFD and previous researches. The listed values will be verified in Phase II.

<table>
<thead>
<tr>
<th>Joint Type</th>
<th>Dry Gap Unfilled</th>
<th>Gap Filled with Compound</th>
<th>Gap Filled with Epoxy</th>
<th>Wet</th>
<th>Monolithic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preparation Type</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>First Cast</td>
<td>Second Cast</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CC</td>
<td>CC</td>
<td>0.00</td>
<td>0.60</td>
<td>0.24</td>
<td>1.00</td>
</tr>
<tr>
<td>CC</td>
<td>UHPC</td>
<td>0.00</td>
<td>0.60</td>
<td>0.24</td>
<td>1.00</td>
</tr>
<tr>
<td>UHPC</td>
<td>CC</td>
<td>0.00</td>
<td>0.60</td>
<td>0.24</td>
<td>1.00</td>
</tr>
<tr>
<td>UHPC</td>
<td>UHPC</td>
<td>0.00</td>
<td>0.60</td>
<td>0.24</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The following calculations present the predicted nominal shear resistance of specimen #6, which is a fluted surface for hardened UHPC against hardened UHPC. The fluted shape has average length of 6 in. for both the valley and the protruded part. The interface surface of the flutes is considered a smooth surface. There are two critical sections proposed in Figure 8.6-3. Sections I and II cross the lower and upper parts, respectively. The theoretical calculations shown below are in-line with the proposed approach and tentative factors. The test results will show if the clamping force provided by the sleeve is enough to engage the strength of the protruded part of the flute. This may lead to proposing minimum clamping stress to validate the theoretical strength.

- Interface plane width \((b_i)\) = 5 in.
- Average Flute Length \((l_i)\) = 6 in.
- Yield Tensile Strength \((f_y)\) = 100 ksi
- Area of Rod \((A_{rod})\) = 0.44 in.²
Sleeve Area \( (A_{sleeve}) \) = 7.0 in.\(^2\)
Number of Pockets \( (N_{pocket}) \) = 1.0 pocket

Sleeve Area Resisting Shear Transfer \( (A_{cv, sleeve}) \) = \( (A_{sleeve} - A_{rod})N_{pocket} \)
\[ = (7.0 - 0.44)(1) = 6.56 \text{ in.}^2 \]

Interface Area 1 \( (A_{cv1}) \) = 23.0 in.\(^2\)
Interface Area 2 \( (A_{cv2}) \) = 30.0 in.\(^2\)

**Considering Section I-I as a Cracking Section:**
Cohesion Factor \( (c_1) \) = 0 ksi
Friction Factor \( (\mu_1) \) = 0.6

Cohesion Factor \( (c_2) \) = 2.6 ksi
Friction Factor \( (\mu_2) \) = 2

Cohesion Factor \( (c_3) \) = 2.6 ksi
Friction Factor \( (\mu_3) \) = 2

Provided Horizontal Shear Strength \( (V_{ni,1}) \)
\[ V_{ni,1} = (c_1A_{cv1} + \mu_1A_{vf1}f_y) + (c_2A_{cv2} + \mu_2A_{vf2}f_y) + (c_3A_{cv,sleeve} + \mu_3A_{vf3}f_y) \]
\[ = [(0)(23.0) + 0] + [(2.6)(30) + 0] + [(2.6)(6.56) + (2)(0.44)(100)] = 183 \text{ kip} \]

**Considering Section II-II as a Cracking Section:**
Cohesion Factor \( (c_1) \) = 2.6 ksi
Friction Factor \( (\mu_1) \) = 2

Cohesion Factor \( (c_2) \) = 0 ksi
Friction Factor \( (\mu_2) \) = 0.6

Provided Horizontal Shear Strength \( (V_{ni,1}) \)
\[ V_{ni,1} = (c_1(A_{cv1} + A_{cv,sleeve}) + \mu_2A_{vf2}f_y) + (c_2A_{cv2} + \mu_2A_{vf2}f_y) \]
\[ = [(2.6)(23.0 + 6.56) + (2)(0.44)(100)] + [(0)(30) + 0] = 165 \text{ kip} \]

Therefore, Predicted Horizontal Shear Strength \( (V_n) \) =165 kip

**8.6.2 Voided Slab Push-off Testing**
It is envisioned to test full scale voided slab to evaluate the proposed design recommendations for the most efficient interface details. The interface details will be finalized depending on the testing results of the push-off components. Figure 8.6.2-1 shows the push-off test setup. The relative displacement between the top part and the lower part will be monitored. To prevent the uplift, vertical reaction through roller will be provided, which allows for translation.
8.6.3 Building Voided Inverted Tee Beam Testing

It is planned to test a voided inverted tee beam for building construction. This type of beam has a good potential to be used in many applications for building construction, and a proposed cross-section for the test specimens is shown in Figure 8.6.3-1.

The test will be performed on a single 60-feet-long beam. The beam will be fabricated and delivered to the laboratory for testing with measurements and observations of camber and cracking made at relevant stages. With the beam supported on a simple span, a 2-inch-thick traditional concrete topping will be cast and cured. The composite action expected to develop between the UHPC beam and the traditional concrete topping is of particular interest. While the topping cures, a series of strain gauges will be installed on the side of the beam and on the side of the topping at various heights on selected cross-sections. Gauges will be oriented to measure strains parallel to the beam length, allowing the strain distribution to be plotted. A discontinuity in the plotted strain distribution at the topping-beam interface would indicate non-composite behavior.

The beam would be loaded via simulated stem loads along the ledge. An initial loading condition may include simulated dead load placed along one ledge only to examine the beam response to unbalanced loading that commonly develops during construction. After unloading, the beam will be loaded along both ledges to the factored level and this level will be held in place for 24 hours. After the 24-hour load test, the beam will be unloaded, allowed to recover, and reloaded to failure. While a flexural failure is expected, it is possible that ledge failure or separation of the topping will control prior to global beam flexure failure. Depending on specimen condition after this failure, additional tests will be performed on the remaining portions of the failed beam. Options for these additional tests include: reconfiguring the supports and testing a segment of the beam in shear; testing the composite deck for punching capacity; and/or testing the ledge locally for punching shear resistance and ledge-to-web attachment capacity. Additionally, a series of tests will be performed on the beam ledges to establish local ledge limit states.
8.6.4 Testing of Partial Depth Bridge Slab Made Composite with CIP CC

One of the demanded products in bridge construction is the partial depth deck. It was envisioned to develop a 1.5 in thick UHPC slab made composite with a CIP CC deck with a total depth of 8 in. The UHPC slab will be cast with steel truss as shown in Figure 8.6.4-1. The slab will be tested in flexure for negative and positive bending moments. These tests will evaluate the slab strength under construction loads. Then, CC will be cast and cured. The composite section will be tested to evaluate the interface shear behavior between the UHPC with unroughened surface and the CIP CC.

![Figure 8.6.4-1. Partial Depth UHPC Slab Details: (a) Plan View, (b) Elevation; and (c) Cross Section](image)

8.6.5 Testing of UHPC Modified NU Bridge Girder Made Composite with UHPC Ribbed Deck

The following section for product development presents a short-term implementation for UHPC in bridge construction. This could be done by modifying web and flange dimensions of the standard conventional I-beams of any DOT to create UHPC I-beam. The top flange may be narrowed significantly and second cast of UHPC ribbed deck will be connected through pockets within the deck. The interface plan will be prepared using the most efficient details concluded from the push-off components. The ribbed deck will be shimmed creating a minimum haunch of 2 in. The haunch and pockets will be cast with UHPC. A 40-ft long composite girder will be tested twice to evaluate the flexural and shear performance of the composite section.
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Interface Shear Behavior

Figure 8.6.5-1. Modified NU Bridge Girder with UHPC Ribbed Deck

8.7 References


9 PUNCHING SHEAR DESIGN

9.1 Notation

\( b_c \) = perimeter of the critical section for shear enclosing the support/concentrated load area (in.)
\( f_r \) = resisting shear stress of UHPC (ksi)
\( f_c' \) = specified strength of concrete at 28 days (ksi)
\( f_{ct} \) = average splitting tensile strength (ksi)
\( f_{pc} \) = average effective prestress around the critical shear perimeter (ksi)
\( h \) = effective depth of the slab around the critical shear perimeter (in.)
\( L \) = length of bearing plate or pad (in.)
\( M_v \) = bending moment transferred from the slab to the support in the direction being considered (ft-kips)
\( V \) = shear force transferred from the slab to the support in the direction being considered (kips)
\( V_n \) = nominal punching shear resistance (kips)
\( W \) = width of bearing plate or pad (in.); length of bearing measured parallel to the direction of the span that produces \( M_v \) (in.)

9.2 Introduction

One of the advantages of UHPC is the ability to significantly reduce the dimensions of the structural members. In addition to the reduction of the total depth and the web thickness of members, the thickness of a solid slab and the ledge could also be significantly optimized. This, in turn, introduces the possibility of punching shear failure for these now thinner elements. Punching shear failure could be an issue for the 2-in. shell of the ribbed slab of a bridge’s decked I-beam, the top flange of a building’s voided slab, and the ledge of an inverted tee beam.

9.3 Previous Work

Many researchers have recognized the importance of the punching shear strength of UHPC. Harris and Roberts-Wollmann (2005) performed punching shear tests on twelve 45-in. x 45-in. UHPC slabs without continuous reinforcement. The intend of this research was to evaluate the punching shear strength of the top flange of a UHPC double tee beam under a wheel patch load. The researchers evaluated this behavior utilizing three different thicknesses (2, 2.5, and 3 inches) with different sizes of punching plates (1-9 in.\(^2\)). All the slabs were clamped at the four edges to prevent the rotation of the slab. Commercial UHPC was utilized with a fiber content of 2% by volume and a compressive strength of 31.6 ksi. Seven specimens experienced punching shear failure, and five failed in flexure. The test results were analyzed and compared against different prediction models (Narayanan and Darwish (1987), Tan and Paramasivam (1994), Shaaban and Gesund (1994), and ACI 318-02 (2002)). It was reported that the punching shear model, Eq. 9.3-1, as specified by ACI 318, provides adequate prediction for the nominal strength of the UHPC. This model assumes a cracking angle of 45° on the horizontal plane and a resisting perimeter located at the mid-height of the slab. A different model was proposed to better predict the nominal strength of the UHPC. The proposed model is an adjusted model for concrete breakout strength, as specified by ACI 318, shown as Eq. 9.3-2. The proposed model assumes a cracking angle of approximately 35° from the horizontal plane, and a resisting perimeter located at the opposite side of loading. Figure 9.3-1 shows a comparison between the measured and predicted strengths, by utilizing ACI318 and the other proposed models. It should be noted that the resisting height of the failure plane is the total height of the slab, not the effective depth from the extreme compression fiber to the centroid of the tensile force since no continuous reinforcement was used. It was also concluded that a 1-in.-thick slab can safely sustain the wheel loads of the AASHTO standard truck, HS20, for bridge applications.

\[
V_n = 0.125\sqrt{f_c'(2W + 2L + 4h)}h \quad \text{Eq. 9.3-1}
\]
\[
V_n = 0.38f_{ct}\frac{(3h + W)(3h + L) - WL}{\sqrt{h}} \quad \text{Eq. 9.3-2}
\]

where
\[
f_{ct} = 1.6 \text{ ksi (Harris and Roberts-Wollmann, 2005)}
\]
Moreillon et al. (2012) tested 18 thin-solid slabs to evaluate the punching shear strength of UHPC. This research tested the effects of the fiber content (1% and 2%, by volume), the ratio of continuous reinforcement (0 to 0.026), and the slab thickness (1.2, 1.6, 2.4, and 3.15-in.). All slabs had the dimensions of 37¾-in. x 37¾-in., and were supported by eight columns in a circular pattern that allowed the slab to rotate. The punching load plate had a diameter of 3.15 in., and was the same for all specimens. It was reported that all the slabs without continuous reinforcement had failed under flexure. Figure 9.3-2 shows the comparison between the measured and predicted strengths, utilizing ACI318 and Harris and Roberts-Wollmann (2005) models, regardless of the continuous reinforcement ratio or the fiber content. These results show that ACI 318 underestimates the punching strength of slabs with a thickness of 3.15 inches. The Harris and Roberts-Wollmann (2005) model provides a conservative prediction for most of the specimens.

Joh et al. (2008) tested six UHPC specimens as part of the Korea Institute of Construction Technology’s (KICT) efforts to evaluate the punching shear strength of UHPC. All specimens have a square shape of 63-in. x 63-in. and a
compressive strength of 28.1 ksi. To prohibit rotation, the slabs were supported from four sides. The slabs have two different thicknesses of 1.6 and 2.75-in., and are loaded with three different plate sizes. The 1.6-in.-thick slabs failed in flexure. The test results were compared against both the ACI 318 prediction model and the Harris and Roberts-Wollmann (2005) model. Both models underestimated the punching shear strength by 25%. The authors concluded that the ACI 318 model provides a reasonable prediction for punching shear strength.

Aaleti et al. (2013) developed design guidelines for precast UHPC waffle deck panels. The waffle slab has longitudinal and transverse ribs and is topped with a 2.5-in.-thick skin. The research team tested the top skin to evaluate the punching shear strength. A 6-in. by 8-in. steel plate was used when loading the specimens to punching shear failure. While the deck skin experienced flexural cracks, which propagated when increasing the load, the failure took place suddenly due to punching shear. The reported failure load was 154.6 kips. A truncated pyramid was separated from the deck, leaving a hole on the contact surface with dimensions equal to the testing plate size. The edges of the pyramid were sloped 45°. Applying Eq. 9.3-1 with a shear perimeter of 28.0 in. and a compressive strength of 26.0 ksi results in a nominal shear capacity of 60.6 kips. The significant difference between the capacity and the demand may be attributed to both the end conditions of the specimens and the close spacing between the ribs.

Gowripalan and Gilbert (2000) developed guidelines for use of commercial grade UHPC in prestressed concrete beams for VSL, Australia. It was proposed that the punching shear strength for slabs at support locations is affected by the global stresses, in addition to the level of prestressing. Eq. 9.3-3 presents the punching shear strength prediction model. In the case of a concentrated wheel load on a non-prestressed deck, \( M_s \) should be placed equal to zero, resulting in a shear strength of 0.725 ksi for UHPC, with a compressive strength of 21.8 – 31.9 ksi. This may be converted to a shear strength of \( 0.13 \sqrt{f_p - 0.16 \sqrt{f_c'}} \). One of the main products was a decked-tub beam. The deck of this product was a waffled slab cast monolithically with the tub. The longitudinal and transverse ribs are spaced at 16.1 in. (410 mm) and 39.4 in. (1000 mm), respectively. The ribs support a deck top skin of 3.15 in. (80 mm).

Equation 9.3-3
\[
V_n = \frac{1}{\left(1 + \frac{b_o M_v}{8V(2L)h}\right)} \left(f + 0.3f_{pc}\right) b_o h
\]

where

- \( b_o = \) the perimeter of the critical section for shear enclosing the support/concentrated load area, in. = \( (2W + 2L + 4h) \)
- \( M_v = \) the bending moment transferred from the slab to the support in the direction being considered, ft-kips
- \( V = \) the shear force transferred from the slab to the support in the direction being considered, kips
- \( h = \) the effective depth of the slab around the critical shear perimeter, in.
- \( L = \) length of bearing measured parallel to the direction of the span that produces \( M_v \), in.
- \( f = \) resisting shear stress of UHPC = 0.725 ksi
- \( f_{pc} = \) average effective prestress around the critical shear perimeter, ksi

9.4 Proposed Tentative Design Recommendations

The previous researches show that the current ACI 318 prediction model for punching shear strength of conventional concrete is still valid for UHPC. It should be noted that the AASHTO prediction model is the same as the model specified by ACI 318. The only change is the perimeter of the critical section. For conventional concrete, the perimeter is defined by straight lines drawn parallel, at a distance \( d/2 \) from the edges of the loaded area (ACI 318). The \( d/2 \) distance was changed to the average slab thickness \( (h) \) in many researches. While this change still shows conservative prediction, in this project, it is recommended to use the critical section as specified by current practices, where \( h \) is used in place of \( d \). This will bring Eq. 9.3-1 to be \( V_n = 0.125 \sqrt{f_p} (2W + 2L + 2h) h \).

It should be noted that punching shear resistance of UHPC L-shaped beam ledges were not studied by previous researchers. It is expected that the ledge of the building components in this research will be subjected to a concentrated or uniform load. In both cases, the punching shear behavior may be different than that proposed in this section. Therefore, a prediction of UHPC ledge behavior will be presented in a separate section.
9.5 Plans for Phase II Testing

9.5.1 Punching Shear Testing for 1-in. Flange

It is planned to perform a series of punching shear tests on the 1-in. top flange of the voided slab as shown in Figure 9.5.1-1. The load will be applied at different locations on the top flange using a hydraulic jack supported by a reaction frame. The hydraulic jack will apply load to a 4-in. by 4-in. loading plate which will be placed on top of a 1/8” thick neoprene pad bearing directly on the specimen. This test will provide evaluation for the proposed recommendation for punching shear strength of UHPC.

![Figure 9.5.1-1. Punching Shear Test Setup for Voided Slab](image)

9.6 Pilot Testing: Punching Shear Testing of 1-in. Lid Slab

A series of tests was conducted on 1-in-thick lid covering 12-ft-long voided slab specimen to determine the punching shear strength of UHPC. The cross section is shown in Figure 9.6-1.

![Figure 9.6-1. Voided Slab Cross Section](image)

The specimen was supported directly on the laboratory floor. In each test, load was applied using a hydraulic jack supported by a reaction frame. The hydraulic jack applied load to a 4” x 4” loading plate which was placed on top of a 1/8” thick neoprene pad bearing directly on the specimen. The jack was operated manually with an electric pump that enabled incremental application of the load. A photo of the test setup is shown in Figure 9.6-2. Specimens were loaded in approximate increments of 2 kips. The applied load was held at each level for the short period of time necessary to examine the specimen and to document/photograph any observations (typically less than 2 minutes at each load point).
A pair of 3" by 6" nominal size UHPC cylinders labeled “top pieces” were delivered with the test beam. Compression tests were performed to determine the concrete compressive strength within 3 days of beam testing. Cylinders were tested in accordance with ASTM C39 and presented average compressive strength of 16.40 ksi.

Five (5) punching tests were completed on a 1” thick UHPC lid slab at different locations. Punching shear failures were achieved in the middle of two stems and adjacent to one stem, relatively far from the edges of the specimen. Tests performed at the slab ends showed more of a flexural type failure mode. A schematic of the approximate failure regions and their respective capacities is presented in Figure 9.6-3. The peak loads at punching failure were 10.8 kips and 10.2 kips. Photo of a typical punching failure is shown in Figure 9.6-4. The average failure load was measured as 10.5 kip. The predicted punching shear strength was 9.1 kip according to the proposed prediction model.
9.7 References


ACI Committee 318. (2002). Building Code Requirements for Structural Concrete and Commentary, (ACI 318R-02), American Concrete Institute: Farmington Hills, MI.


10 MISCELLANEOUS

10.1 Ledge Design

10.1.1 Introduction and Background

The proposed building framing system in this project consists of 60-feet-span hollow inverted tee beams supporting 60-ft-span double tee beams and voided slabs for parking and offices buildings, respectively. No previous work has been conducted on the behavior of UHPC ledges. Therefore, the research team has proposed some conservative assumptions as a starting point to design the ledges based on the behavior UHPC exhibits under different stress states. Preliminary design has been performed to provide some options for detailing the ledge of the 60-ft hollow inverted tee. These options were evaluated by Finite Element Analysis (FEA), and the final details will be tested in Phase II.

The PCI Design Handbook (8th Edition) reports that previous Handbook editions overestimated the punching shear strength of the conventional concrete ledges. Based on Rizkalla et al. (2016), the PCI Design Handbook specifies a design procedure applied to ledges with: (1) heights, $h$, of 8 to 18 in., (2) projections, $l_p$, of 6 to 10 in., (3) bearing widths, $b$, of components on the ledge ranging from 4 to 12 in., (4) Distance of the end load to end of ledge, $d_e$, ranging from 4 to 36 in., and (5) design compressive strength, $f'_c$, of 5.0 to 15.0 ksi. However, it was reported that the procedure may be applicable for different ranges.

The PCI Design Handbook (8th Edition) specified a minimum punching shear strength of $\sqrt{f'_c}$ (in psi) for normal weight conventional concrete ledges of non-prestressed components subjected to concentrated loads. The shear strength of the ledges could be improved if the factored load-to-nominal strength ratio is 0.6 or less. This global effect is considered by multiplying the minimum shear strength by factor $\beta$, ranging from 1.0 to 2.0. The prestressing force also improves the shear strength of the ledge. The prestressing can increase the shear strength by $\gamma = \sqrt{1 + 10 \frac{f_{pc}}{f'_c}}$, where $f_{pc}$ is the prestressing force after losses, divided by beam gross area. The shear strength of ledges supporting continuous loads was specified to be $\sqrt{f'_c}$ (in psi). It should be noted that this was reduced from the previous PCI Design Handbook (7th Edition) by 50%, which was concluded from test results of concentrated load programs. An extensive project was performed on ledges supporting concentrated loads to develop Eqs. 5-76 to 5-79, as shown in PCI 8th Edition, with supporting research by Rizkalla et al. 2016. These equations were classified differently than the equations for ledge design present in the PCI 7th Edition Handbook to provide a rational design method for beam ledges.

Rizkalla et al. (2016) found that the punching shear cracking path inclined at a slope of 1:2 (vertical:horizontal) when the transverse reinforcement was uniformly spaced over a width of $6h_s$, see Figure 10.1.1-1. However, concentrating the ledge reinforcement creates an angle of 45 degrees, as assumed by the PCI 7th Edition Handbook. Cracking of the UHPC ledge may follow the flatter angle due to the random distribution of the fibers.
A ledge must also resist any tensile stresses from the cantilever behavior. The conventional concrete ledge is reinforced with transverse flexural rebars, $A_s$, to resist the bending moment created by the applied load eccentricity, $a$, from the inside face of the beam. Figure 10.1.1-2 shows typical details for ledges of conventional concrete L-beams. This reinforcement is placed over a distance of $6h_l$ on either side of the bearing, but is not to exceed half of the distance to the next load, $s$. This reinforcement may be eliminated in UHPC ledges as long as the fiber contribution to the tensile strength exceeds the factored loads, with a sufficient factor of safety.

Conventional concrete ledges are reinforced with longitudinal rebars, $A_b$, providing a minimum longitudinal bending strength of 0.20 ksi to the ledges. Calculations have shown that this reinforcement is not needed for UHPC, which has a minimum flexural cracking strength of 1.5 ksi.

Hanger rebars, $A_{bh}$, are provided to attach the ledge to the web of the conventional concrete beam. These rebars prevent horizontal cracking at the top of the ledge, which can develop and propagate through the web of the beam. This horizontal crack is indicative of the forces acting to separate the ledge and the bottom part of the web from the remainder of the beam. PCI specifies that hanger bars should be placed over a distance of $6h_l$ on either side of the bearing, but not to exceed $s/2$. Raths (1984) proposed Eq. 10.1.1-1 to determine the required hanger rebars. Figure 10.1.1-3 shows the hanger steel details, as proposed by Raths (1984). This procedure was considered overly conservative, as reported by Klein (1986). The current PCI equation for hanger steel was developed by Klein (1986).
It takes the internal shear stress distribution and torsional strength of the overall beam into account. This reduces the demand for steel hanger. Conservatively, Raths approach may be used for a UHPC ledges.

\[
A_{sh} = \frac{V_u (b_w + 0.75 l_p - a)}{\phi f_y (b_w - d's - a/2)}
\]

Eq. 10.1.1-1

where

- \(V_u\) = applied factored load (kips)
- \(b_w\) = web width (in.)
- \(0.75 l_p\) = applied load eccentricity from the inside face of the beam (in.)
- \(a\) = compression block depth from flexural strength analysis (in.)
- \(d's\) = distance from centroid of hanger to the inner web face (in.)
- \(f_y\) = yield strength of hanger steel (ksi)

**Figure 10.1.1-3. Hanger Steel Details (Raths, 1984)**

### 10.1.2 Proposed Tentative Design Recommendations

Previous research and pilot testing conducted on the punching shear strength of UHPC reported similar behavior to that of conventional concrete. Therefore, the PCI approach for punching shear analysis is proposed for UHPC ledges with one conservative change. This change is related to the global effect on the ledge behavior. The \(\beta\) coefficient of shear strength may be always placed equal to 1.0.

The bending (cantilever) behavior should be analyzed by considering the tensile strength of UHPC of 0.75 ksi. Current PCI provisions specify an effective width of \(6h_l\) on either side of the bearing, but not to exceed \(s/2\). It is tentatively proposed to apply this provision to UHPC ledges. A moment-curvature diagram should be developed to determine the ultimate flexural capacity of the ledge without transverse reinforcement. In this case, the nominal strength should be reduced using a strength reduction factor of 0.5. A rectangular cross section should be considered with a width and a total depth of \(12h_l\) (but not more than \(s\)) and \(h_l\), respectively.

The first cracking strength of UHPC is specified at a level of at least 1.5 ksi in this research. This level exceeds the current minimum requirement for longitudinal bending reinforcement of 0.2 ksi. It is proposed to drop the longitudinal bending reinforcement requirement when using UHPC.

It is proposed to determine the flexural strength of the web without rebars, with an effective length of \(6h_l\) on either side of the bearing, but not to exceed \(s/2\). The nominal flexural strength, \(M_n\) of the web should be determined by developing a moment-curvature diagram for UHPC with a tensile strength of 0.75 ksi. The reduced flexural strength, \(\phi M_n\), shall not be less than the factored applied moment, \(M_u\). The strength reduction factor is 0.5 for sections without hanger rebars. The factored applied moment, \(M_u\), may be calculated as the applied loads times its eccentricity from the remote face.
10.1.3 Finite Element Analysis

3D non-linear finite element analysis was performed for UHPC hollow inverted tee (IT) specimens using the commercial software ATENA. Material parameters were calibrated using the average results of available third-point bending test data for the specific mixes created for this project by multiple precasters, as detailed above. A total of two proposed IT cross-sections were analyzed. The purpose of this analysis was to determine the ultimate ledge capacity, to assess the feasibility of the proposed cross-sections. Details of the finite element model are summarized in this section, along with the results.

Two hollow 20'-0" long UHPC IT beams were analyzed in this program. Figure 10.1.3-1 shows details of the two proposed ITs. These sections were detailed based on the proposed tentative design recommendations. It should be noted that the analysis has been conducted using a tensile strength of 1.25 ksi (derived from inverse analysis), not 0.75 ksi. The design will be revised to account for the new recommendations in Phase II. The first section has a ledge height of 6 inches. The 6-in-deep ledge was enough to resist the factored moment from the stem of the 60-ft-span double tee, for a parking structure without transverse rebars. The calculated factored load of a typical 12DT30 double-tee beam was 27.8 kips. The second section has ledge height of 3 inches. The reduction in the ledge height has reduced the flexural strength and led to a need for transverse reinforcement of No. 3 @ 6 in. Both sections had enough capacity to eliminate the steel hangers. The sections also had discrete steel reinforcement in the bottom slab, and was located between the webs. Steel No. 6 bars were chosen as the longitudinal reinforcement to prevent global flexure failure.

![Figure 10.1.3-1. UHPC IT Cross-Section with 6" Ledge (on Left) and 3" Ledge (on Right)](image)

A fracture-plasticity material model, with a user-defined tensile function, was used within ATENA to model the UHPC. The material model was calibrated to existing third-point bending test data and other material data provided by Wiss, Janney, Elstner Associates, Inc. (WJE). The calibration was performed using an ATENA-simulated third-point bending test, along with a method of inverse analysis to produce the final results. The inverse analysis shows an actual tensile strength of 1.25 ksi. This strength is higher than the minimum specified tensile strength in this project. The FEA will be revised in Phase II to reflect the minimum specified tensile strength. An option under consideration for FEA tensile-model modification include picking a reduced first cracking tensile strength while keeping the same post-cracking tensile function. This change would reduce the ultimate tension strength in parallel with the cracking strength. Alternatively, the ultimate tensile strength could be maintained by adjusting the tension function to match the selected cracking strength and ultimate strength. Additional simple physical experiments in tension may be needed to refine the tension material model. The calibration procedure for the material model in flexure generated a near perfect match between the numerical model and the simple 3-point bending experiments. The calibrated material model was then deployed on the other analyses described in this report.

A schematic of the numerical test setup is shown in Figure 10.1.3-2. Note that for the test set-up within ATENA, only one plane of symmetry was utilized for an increased computational speed. Therefore, only half of the 20'-0" UHPC IT specimen is modeled. The specimen was analyzed with simple supports. Steel plates were used at all bearing locations to mimic a laboratory test and to prevent unrealistic stress concentrations at the load points and reactions. The analysis was performed using step-wise displacements applied to a loading plate. The applied force was monitored along with the vertical deflection of the bottom face of the ledge directly underneath the applied load.
For the section with a 6-in ledge, the FE model displayed concentrated vertical tension strains in the web at a load of 33.7 kips. This load level is approximately 6 kips higher than the calculated in-service load of a typical 12DT30 double-tee beam. Strains were greatest at the web-to-ledge interface. However, the maximum vertical strain of 0.000206 is well within this UHPC's capacity. The observed failure mode appears to be a hanger failure in the web. The web was observed to separate along a horizontal crack at a location above the ledge, at a load of 89.7 kips. The cracking at this interface propagates along the length of the specimen. Figure 10.1.3-3 shows this separation from the inside of the web, where it can be clearly seen that it occurs at the end of the transition portion of the section that tapers from between the flange and web. The results show cracking throughout the specimen, but a lot of this cracking is minor. It was seen that all damage is concentrated around a primary horizontal crack in the web, at a location slightly above the ledge.

Figure 10.1.3-2: Schematic of the Test Setup for Simulation (6 in. Ledge Shown)

For the section with a 3-in ledge, the FE model displayed significant tension strains (~0.002) in the web (vertical tension) and in the positive moment region of the ledge (longitudinal tension). Strains were also observed in the negative moment region of the ledge (tension in a direction parallel to the ledge projection). Minor cracking was observed at the web-to-ledge interface. The applied load of 30.6 kips, at this stage, is approximately 3 kips higher than the calculated in-service load of a typical 12DT30 double-tee beam of 27.8 kips. The observed failure mode appears to be mainly caused by a direct tension failure at the web-to-ledge interface, at a load of 67.2 kips. This observation is based on the fact that the highest strains are in the vertical direction at this location, see Figure 10.1.3-4. There was also significant strain induced by negative and positive bending in the ledge, as well as high
shear strains. Therefore, this failure is likely due to an interaction of tension, punching shear, and bending behavior of the ledge and web.

\[ \text{Figure 10.1.3-4. Vertical Strains in the Web and Ledge at a Ledge Load of 67.2 kips} \]

In summary, the main failure mode in the test of the 6 in. high web appears to be a hanger mode in the web. The main failure mode in the test of the 3 in. high web appears to be a ledge-to-web attachment mode, possibly including a shear failure at the ledge-to-web interface. The 3 in. ledge displayed significant distress in both bending and shear, and the failure modes are likely an interaction of multiple behaviors. Both designs were sufficient to withstand the expected in-service load of a typical double-tee beam. However, both designs, especially the 6 in. ledge, would likely be enhanced from additional hanger reinforcement, particularly at load locations. This finite element study will continue, studying different cross-sections and different reinforcement schemes.

10.1.4 Plans for Phase II Testing

The ledges of UHPC beams must resist a variety of local stress states. These states will be investigated by loading the ledges of the hollow inverted tee beams. The beam ends will be supported against torsional rotation as needed, see Figure 10.1.4-1. Loads can then be applied to the ledge individually, or in selected pairs or groups to study the punching shear behavior of the ledge, the ledge attachment behavior, and the ledge hanger behavior.

\[ \text{Figure 10.1.4-1. Conceptual Ledge Behavior Test Setup} \]
Two 20-feet segments will be fabricated for ledge testing. Two different geometries (for example, a 3-inch deep by 8-inch projection and a 6-inch deep by 8-inch projection) will be tested. The ledge reinforcement will be varied at different points along the four ledges (two ledges on each 20-feet segment). Different ledge-web attachment details could be tested (including no discrete attachment), and different levels of hanger reinforcement could be tried (including no hanger reinforcement).

The number of ledge tests that can be obtained from each 20-feet segment will depend on the extent of the various failure modes that develop. It is anticipated that a minimum of three tests per 20-feet length of ledge can be achieved, for a total of at least 12 ledge tests. Figure 10.1.4-2 shows details for the testing plan for the hollow inverted tee with 3-in. and 6-in. ledges.

![Figure 10.1.4-2. Testing Details for Ledge Behavior](image)

10.1.5 References


10.2 End Zone Stresses

10.2.1 End Zone Bursting Reinforcement

Web end cracking is a significant phenomenon in highly pretensioned concrete members with large bottom flanges, when prestress dominates, and thin webs required in efficient members. It has been the subject of much research, see for example Marshall and Mattock (1962), Tadros et al (2010), and Okumus and Oliva (2013). Members optimized for UHPC experience more extreme pretensioning forces and web widths. However, the superior tensile capacity of UHPC provides an offsetting effect.

This project will not address end zone cracking as a primary topic of research and possible revision of the estimated design bursting force. Rather, it is focused on application of the force currently specified in the AASHTO specifications, which is 4 percent of the prestressing force, and apply it to an end zone that has two resistance elements, rather than just the bursting rebar in conventional concrete. The second resistance element is the tensile capacity of the UHPC.

As given in other sections of the report, the UHPC used in this project is expected to have minimum tensile strength and ductility requirements. Specifically, the minimum flexural cracking strength measured in accordance with ASTM Standard C1609 is 1.5 ksi and the minimum ultimate strength is 2 ksi. Other requirements related to ductility in tension are also given. It is assumed for design for bursting the tensile capacity of concrete is 0.75 ksi. This is one-half of the minimum flexural cracking strength of the concrete, and is very conservative at this time. The research team understands that FHWA researchers are recommending a stress limit of 1.00 ksi for this check (unpublished yet). However, we believe that the average over the assumed tensioned area of (h/4), where h is the total member depth should be approximately one-half of the peak stress at the very end of the member. Gowripalan and Gilbert (2000) recommend a value of 5 MPa (0.73 ksi) for the average stress to be used in the design for bursting forces in UHPC, which is consistent with our recommendations.

The design philosophy currently in the AASHTO is to limit the stress in the bursting reinforcement to a maximum of 20 ksi in order to keep the bursting crack width to a minimum. Also, as shown in Tadros et al (2010), the most effective rebar is the bar closest to the end of the member. Accordingly, the recommendations used for design of UHPC members to control (not eliminate) bursting cracking is as follows:

Calculate the prestress force just after release to the precast member = initial tension minus estimated initial losses. Only strands that are bonded at the member end need to be counted.

Calculate 4 percent of the initial prestress

Multiply the web area, (h/4*b)_w, by the 0.75 ksi average stress over that area to obtain the concrete contribution to resistance of the bursting force.

The net force from step 2 less that from step 3 is divided by 20 ksi to determine the required steel rebar. Place the rebar as close as possible to the end of the member. The bars should be as close to each other to effectively control end cracks, yet far enough from each other to allow for good fiber distribution. A clear bar spacing of 2 inches for the ½” and ¾” fibers used in this project would ensure no interruption of fiber passage between the bars.

The examples in the Appendix illustrate how this design recommendation impacts the beam end detailing.

10.2.2 References


10.3 Vibration

There are different sources of vibrations on building floors. The vibration may be generated by walking, rhythmic activities, mechanical equipment, or vehicles. There are limited resources on the human response to floor vibration for concrete floor systems. AISC Steel Design Guide 11 has addressed the floor vibrations due to human activities. This may be considered as the base of the design guidelines provided by PCI Design Handbook. The vibration effect could be neglected in the design process of concrete floors due to the large stiffness of the floor. However, it may govern the dimensions of UHPC cross section because of the shallower and thinner proposed components.

The process of evaluation a floor system subjected to vibrations caused by walking is to compare the fundamental frequency of vibration with the minimum fundamental frequency. The fundamental frequency of vibration for floor system is a fraction of gravitation acceleration. It is also highly dependent on the span length, mass, stiffness and material properties. The following calculations present an example for checking the vibration caused by walking of voided slab system of office building.

Given:
Open office area: 60 ft by 60 ft
12-feet wide voided slab panels
Weight of panel = 52 lb/ft²
Moment of Inertia = 47,540 in.⁴
Static modulus of elasticity of UHPC, E = 6,000,000 psi

Solution:
Minimum fundamental frequency, \( f_{o \text{ min}} \), of the floor system could be calculated as

\[
 f_{o \text{ min}} \geq 2.86 \ln \left( \frac{K_w}{\beta_m W_f} \right)
\]

where
\( \ln \) = natural logarithm and the constant 2.86 has the unit Hertz (cycles/second)

\( K_w \) and \( \beta_m \) are factors to account for human perception of vibration, where

\[
K_w = 13.0 \text{ kip} \quad \text{(PCI DH Table 12.6.1)}
\]

\[
\beta_m = 0.02 \quad \text{(PCI DH Table 12.6.1)}
\]

\( W_f \) = effective weight due to self-weight and some of the live load over the effective width of B, kip

Estimate effective weight, \( W_f \):

\[
w_t = 52 \text{ lb/ft}^2 + 10 \text{ lb/ft}^2 = 62 \text{ lb/ft}^2 = 0.062 \text{ kip/ft}^2
\]

It is not only the flexural stiffness affecting the response of the floor system under vibration but also the torsional stiffness. For systems with higher torsional stiffness, it is recommended that B equal the span. Thus

\[
W_f = w(B)(l)
\]

\[
= 0.062(60)(60) = 223 \text{ kips}
\]

Therefore,

\[
f_{o \text{ min}} = 2.86 \ln \left( \frac{13.0}{0.02(223)} \right) = 3.08 \text{ Hz} \geq 3.0 \text{ Hz}
\]

The expected fundamental frequency, \( f_o \), of the floor system could be calculated as

\[
f_o = \frac{\pi}{2l^2} \sqrt{\frac{gE}{w}}
\]

where
\( l \) = panel span = 60 ft
\( g \) = gravitational acceleration = 386.4 in./second²
\( w \) = panel weight = (52)(12) = 624 lb/ft = 52 lb/in.

Therefore,
\[ f_o = \frac{\pi}{2(60)^2(12)^2} \sqrt{\frac{(386.4)(6,000,000)(47,540)}{52}} = 4.4 \text{ Hz} \] > \[ f_o \text{ min.} \]

Thus, the floor system satisfies vibration recommendations.

### 10.4 Handling and Shipping Considerations

#### 10.4.1 Notation

- \( K_\theta \): rotational constant of the spring support
- \( \alpha \): super elevation of the roadway supporting the vehicle
- \( \theta \): rotation angle of the girder from vertical
- \( W \): weight of beam
- \( M_r \): resisting moment to overturning from wind and centrifugal force during transit
- \( z \): lateral deflection of the girder under self-weight
- \( e_i \): initial lateral eccentricity of the center of mass of the girder with respect to the roll axis or center
- \( y_r \): distance from the roll axis/center to the center of mass of the girder

#### 10.4.2 Introduction and Background

Due to the regulations on the shipping height, deep girders would not be permitted to be shipped on many city roads. The proposed 9 ft deep bridge beams are viable for shipment by barge. Concrete Technology Corporation, Washington, and Standard Concrete Products Company, Florida and Georgia, have made 12 to 15 ft deep pier segments for 250-300 ft spans when shipping by water is possible. For shipment by truck, 8 to 9 ft is perhaps the deepest section that can be used, unless overhead restrictions are scouted in advance for the route being considered. For the design examples in this research, maximum depth of 9 ft is used to allow for road shipping.

Girder stability basics have been introduced by Mast (1989, 1993). PCI (2016) published a recommended practice for lateral stability of precast, prestressed concrete bridge girders. PCI incorporated Mast’s approach and added additional criteria to consider the lateral stability of the girder from bed to bridge. This also includes the bracing requirements during the construction of the new deck or the replacement of the existing deck. The precast prestressed members should have a certain factor of safety against cracking and overturning. This should be valid during all construction stages starting from lifting the member after stripping ending with providing the permanent lateral supports. All stages should be checked.

#### 10.4.3 Stability of Decked I-Beam

The wide top flange of a 250-ft Decked I-beam may lead to a risk of rolling over during transportation or at case of single girder on bearing. These cases should be carefully addressed in addition to all other cases. Figure 10.4.3-1 shows the rotated beam on hauling system. The overturning moment is caused by beam’s self-weight, wind load, and centrifugal force. The resisting moment is provided by the stiffness of transport rig. The super-elevation of the roadway has a significant effect on the overturning and the resisting moments. The factor of safety against rolling over shall not be less than 1.5. Girder Stability Analysis V1.0 program was utilized to evaluate the safety against cracking and overturning of the decked I-Beam. The rotational spring constant of the shipping support and center-to-center wheel spacing were assumed 80,000 kip-in/rad and 96 in, respectively. This represents the largest rig we currently know it is available. The required factor of safety against rollover was achievable by using overhang of 36 ft. This overhang at the front end may cause a problem with the girder interfering with the cab, but that could probably be resolved.
While stability analysis shows possibility of transporting the most critical product in this project, it shows high demand on lateral bracing during construction. Once the girder is placed on the permanent bearing devices, it should have a sufficient factor of safety against cracking and rolling over due to lateral forces. The spring constant of the bearing device is considered a significant factor for the stability of the girder at the final position. The analysis shows a need for lateral bracing system once the girder is placed on the bearing and the crane rigging is released to avoid cracking and rollover possibilities.

10.4.4 Summary

The available knowledge and resources used for conventional concrete products to study their stability from bed to bearing are still valid for UHPC products. The available hauling vehicles can deliver the products of this research safely. Lateral bracing may be provided at the time of placing the bridge girders on the bearing devices during active and inactive construction. To extend the bridge girder length further, it is possible there is a larger roadworthy vehicle somewhere else, but the research team don’t know any details.

10.4.5 References


Precast/Prestressed Concrete Institute (PCI). 2016. Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders. PCI, Chicago, IL.
10.5 Web Buckling

A separate section addressed the stability of the member during handling and shipping. In addition, it is possible that a beam with a very thin web could buckle during handling, or more likely due to full gravity loads.

Web width in UHPC beams is primarily controlled by shear strength requirements. The web width is generally narrower than in conventional concrete members because the shear strength of UHPC is so high as to not require stirrups or large web widths. Therefore, there may be concern about web buckling. For example, the web width of the decked I-beam recommended for use in this project is only 4 inches for a 9-ft tall web. Several studies have been performed to examine web slenderness in UHPC, see Abrams (2013), Krahl et al. (2016), and Rosso et al. (2015).

The conditions for which the web slenderness should be checked are:

1. During lifting off the precasting bed
2. During shipping
3. During handling at the jobsite
4. Placement on the bearings
5. Before the deck (or longitudinal joints in decked I beams) is made composite
6. Due to full load

During shipping and erection, the beams are subjected to their self-weight, with no lateral bracing. This will affect the unbraced length of the web for some shapes. This case must be addressed carefully. Web buckling will not be as critical as in the case of full loading at service. When the beam is placed on the permanent supports, it is possible to have web buckling due to construction loads and before the end diaphragm is constructed. To illustrate the case of a beam bearing on the final bearings before it is braced, a finite element analysis was conducted. **Figure 10.5-1** shows the deformed shape when the edge of the beam is loaded with an additional load beside its own weight. The analysis shows that it takes a load of 222 kips/ft to create a critical buckling load. This is an extremely high load on a somewhat unrealistic system. It is strongly advised that the beams are connected immediately with cross bracing and then followed shortly with concrete diaphragms at the support.

![Figure 10.5-1. Deformation Due to Critical Load at Buckling of 226 kip/ft Applied to the Right Edge](image)

If a heavy construction load is placed at midspan after the end diaphragms are constructed and before the longitudinal joints are cast, then there is a possibility of the web being subject to bending moments.

Based on this limited study, it appears that conventional construction practices and the corresponding stability checks are adequate for the products proposed in this study. The team will spend more time to theoretically investigate this topic and to issue recommendations relative to web and flange thickness as controlled by buckling.
10.5.1 References


11 PRODUCT DEVELOPMENT

11.1 Structural Advantages of UHPC
UHPC has special characteristics that need to be capitalized on as new optimized products and systems are developed. The most significant component for the applications considered in this study is the presence of 2 percent by volume of steel fibers having high tensile strength, high modulus of elasticity, high bond strength (represented by very small diameter and relatively large length/diameter ratio). The presence of such fibers has the following direct impact on structural properties:

(a) High tensile cracking strength which affects service load analysis for flexure at midspan and ends of prestressed members

(b) High tensile ductility which allows for use of some of the tensile strength in flexural strength design, especially in the transverse direction of wide top flanged members

(c) High tensile ductility which allows for improved behavior in shear after cracking due to overloads and for the ability to accept shear design without shear reinforcing bars.

(d) High tensile strength and post-cracking strength and ductility which allows for reduction, or possible elimination, of bursting reinforcement at the ends of prestressed members.

(e) Very high interface shear capacity, especially when fluted (corrugated) interface is used.

The high compressive strength is also valuable in increasing the resistance to large bending moments with relatively small compression flange and the resistance to high shearing forces with very narrow webs.

Additional features that are important but have little impact on sizing of precast concrete products are very high durability and relatively small, prestress loss and camber growth.

11.2 General Guidelines for Optimizing Conventional Concrete Member Dimensions
The criteria used by Tadros and Geren, (1994) to develop the popular NU (Nebraska University) I-Girder shape for bridges, and by the various state highway agency I-beam shapes that followed, are still applicable as the basic criteria for optimization of section cross section dimensions. They are summarized as follows:

(a) Large wide bottom flange. It allows for placement of the largest possible number of pretensioning strands, and thus the highest level of prestress and potentially the longest possible span. Also, a wide bottom flange provides for good stability of the beam during handling, shipping and placement on the supports before the girders are connected with diaphragms and the deck.

(b) Wide thin top flange. It allows for resistance to girder buckling during handling, reduction of side-sweep potential. It reduces the effective deck span in the transverse direction. It helps increase the eccentricity of the prestress and its effectiveness under service load conditions. Using a large top flange does not offer much value under final service conditions as the deck is effectively the "compression flange."

(c) Narrow web. In some cross sections, the web occupies over 40 percent of the total cross section. With modern shear design provisions in the AASHTO LRFD Bridge Design Specifications, the nominal shear strength is allowed to be as high as 25 percent of the concrete compressive strength. Thus, for 10 ksi concrete, Shear strength can be as high as 2.5 ksi. This is nearly three times the earlier AASHTO Standard Specifications and the current ACI 318 Code provisions. One condition for this shear capacity to be achieved is to have an adequate tension tie which is typically provided by the flexural reinforcement at the beam end. Accordingly, it has been possible to have very large, 140-180 ft, spans with a spacing as large as 12 ft, using beams with only 6 in. wide webs. The web width is generally controlled by space availability for two columns of strands and two lines of shear reinforced. For example, 2-0.6” strands plus 2#5 bars would require a web width of 2.6 in. (strand out-to-out spacing) +5/8 in. vertical bar+5/8 in. vertical bar+1 in. cover+1 in. cover = 5.85 in. Obviously, when elimination of shear reinforcement is possible with UHPC and when no strand draping is required, the web width could be reduced significantly to whatever is required for concrete placement and preventing web bucking.
11.3 Additional Considerations for UHPC in Optimizing Member Dimensions

The same criteria can be applied to box beams and U-beams. In both cases, a top flange would significantly enhance the efficiency of the member. This is why a top "bulb" is often provided in U-beams.

An exception to the above criteria is the very popular double tee member. In this case, lack of bottom flange allows for a very rapid production of a product that can cover a floor area of, for example, 12 ft wide by 60 ft long, using fixed forms and relatively light member. It has served its purpose well for parking structures and roof framing applications. But it would be difficult to employ with UHPC due to lack of strand space near the bottom fibers and relatively large cambers and deflections. This is why no attempt has been made in this project to optimize the already optimal double tee joist in parking structures.

Another except to the criteria is the use of rectangular, inverted tee and L-shaped beams to support DT in parking structures and to support slabs in office and residential structures. For these products, which generally carry relatively heavy load on a relatively short span, the wide webs used in these products can be replaced with a voided section with two narrow webs.

The criteria used in this project for UHPC are similar to the general approach used with conventional concrete as outlined above. The following additional criteria have been employed:

(i) Aim to use only 50 percent of the conventional concrete volume. Preliminary cost analysis has revealed that it is feasible to have UHPC superstructure products that are competitive with conventional concrete system assuming that the UHPC fabricated product cost per cubic yard is twice that of the corresponding conventional concrete product.

(ii) Use comparable structural depth to that of conventional concrete products. The moment of inertia, stiffness, live load deflection and vibration would still be more critical with UHPC due to the smaller area and mass. But, the expected shape is likely to satisfy design criteria related to stiffness with a relatively generous structural depth.

(iii) Use a minimum web width of 2 in. with fibers that are at least ¾” (20 mm) long. Narrower webs are possible with shorter fiber. For deeper members and single web primary members (such as bridge I-beams), a minimum web width is conservatively taken as 4 in. to allow for some end zone reinforcement and to protect against possible web buckling.

(iv) Use 0.7 in. strands where possible at a spacing of 2 in. vertically and horizontally. This strand has a cross sectional area of 0.294 in.² which is 35 percent higher than that of 0.6 in. strand and 92 percent higher than that of 0.5 in. strand. It has been found by Morcous et al (2011) to have comparable strand transfer and development lengths to those predicted by current codes for the smaller strands. This choice would allow the member bottom flange for large spans to be reasonably small. For, building members, it may not be necessary to use 0.7 in. strand. For these cases 0.6 in. strand would be the typical product. It is not anticipated that 0.5 in. strand would be widely used with UHPC.

(v) As given in the Materials Guide, method of concrete placement affects the quality of the product. Discharging concrete at the bottom of the member, at the center of the length allows for better fiber distribution and for avoidance of cold joints. Therefore, voided sections, should allow for tremie concrete placement through a vertical void in the insulation void forms.

11.4 Development of 60-ft Span Voided Slab for Multistory Buildings

Currently, the most common floor product for multistory residential and office buildings is the hollow core (HC) plank. Its production is highly automated and its cost is quite reasonable. The most common of the HC plank is the 8 in. deep one which spans about 30 feet. There are deeper members up to 20 in. deep. But, their use in the US is limited due to their heavy weight and lack of demand. Further, it is often difficult to access the voids in a HC plank for housing of utility conduits.

Several initiatives have been pursued in recent years to develop a 60 ft span, 24 in. deep, 100 psf superimposed prestressed concrete floor member. Finfrock Industries, https://finfrock.com/finfrock-news/resources/, has successfully developed the “DualDeck” slab and has successfully used it on a number of projects. It consists of two 2 ½ in. slabs spaced vertically 24 in. Each slab is prestressed with 0.5 in. strands and also reinforced with rebar.
The two slabs are connected with bars and steel angles in the vertical space. Also, the space is used to house utilities in the precast plant. Metromont has also developed “MetroDeck”, http://www.metromont.com/wp-content/uploads/2016/05/MetroDeck.pdf. It is a voided product with a full bottom flange, webs and a partial top flange. The Product is made in one step, compared to the two-step concrete placement method used on the Dual Deck. A third product has been developed by Clark Pacific Company on the West Coast, https://www.clarkpacific.com/product/clark-access-deck-modular-precast/. It is called the “Clark Access Deck.”

The goal here is to develop an optimized UHPC voided slab that has a depth of 22 in. and that can span 60 feet or more while carrying a superimposed live load of 100 psf.

(a) Initial Section Geometry:

\[ \text{Figure 11.4-1 illustrates the initial attempt at optimizing the cross section without consideration for method of production.} \]

\[ \text{Figure 11.4-1. 12-ft Wide by 60-ft Span Optimized UHPC Voided Slab for Multistory Residential and Office Applications} \]

The thickness of the top and bottom flanges is assumed to be 1 in. based on analysis and on observation that the conventional concrete Dual Deck product has successfully been made with 2 ½ in. top and bottom flanges. The top slab must be able to span 3 feet between stems and must have adequate punching shear capacity. The bottom slab provides a flat soffit which may be used as a ceiling in apartment and hotel applications. It is also needed to enhance the flexural stiffness of the product. The one-inch thickness is, however, a challenge if production requires that the relatively sticky UHPC is placed around the void forms and is required to travel to fill the bottom flange.

The web width is 2 in. which is adequate for placement of UHPC with 20 mm (3/4 in.) long steel fibers, and even better with 12 mm (1/2 in.) fibers which are preferred for this application with very thin walls. In the center 60 percent of the slab, it is proposed to place voids as shown in Figure 11.4-1. Previous research by Saleh et al. (1997), which was adopted in the PCI Handbook, demonstrated that such generous openings in a double tee did not cause a detrimental effect in their behavior. Their primary function is to allow for utility conduits to pass through in two-directions. However, if EPS void forms are used in production of the member, it is difficult to remove the EPS blocks to create both the longitudinal voids and web block-outs.
Preliminary analysis shows adequacy of the product for up to 73 ft span. For 60 ft span, which is the focus of this study, prestressing required at the bottom bulb of each stem is 6-9/16" (or 4.6") strands with two strands required in the top bulb of each stem. A single strand should be placed in the edge bulbs. No reinforcing bars are required except, perhaps at the ends for bursting stresses and to enhance the tension tie capacity at the supports.

The product shown in Figure 11.4-1 lends itself to some form of extrusion for production efficiency. It only has strands and fiber reinforced concrete; no bars to be detailed and placed during production. However, at this time, the authors know of no available extrusion methods for UHPC of the very thin dimensions given in Figure 11.4-1.

A two-part production method is proposed as explained below. The two halves can be cast with the same ½ section form. Figure 11.4-2 shows the bottom half as cast and before it is turned 180 degrees to receive the top half. The advantage of this precasting solution is ability to ensure good concreting of the very thin top flange. Note here that only 6 ft wide section is shown for illustration. It is the same section as the one that will be used in Phase II testing in this project. The web block-outs are easier to form. Connection between the top and bottom halves take advantage of the excellent horizontal shear capacity of UHPC with fluted (shear key) interface. Flattened post-tensioning ducts, filled with UHPC through the interface, completes the connection. One disadvantage of this production method is the unconventional step of rotating the bottom half of the product.

An attempt was made by Tindall Corporation while performing mix design and production trials. It was attempted to produce a version of the voided slab product using a two-casting solution. Figure 11.4-3 shows a cross section of the bottom part. The shaded areas represent block-outs formed with expanded polystyrene blocks. All required prestressing was placed in this part of the section. They are 6-9/16" bottom and 2-9/16" top strands in each stem. One-time use wood forms were used to form for the product. The second part is 1 one-inch thick by 4 ft wide top slab, made separately on the bed, with block out locations that matched those at the top of the first part. The block-outs were spaced at 4 ft on centers. This production attempt was successful, despite the difficulty in ensuring that concrete totally filled the 1-inch bottom flange. Also, wood forms did not produce the type of accuracy expected for this product and had to be destroyed to remove the product. Assembly of the two pieces and placement of UHPC in the openings created between the two parts of the section were completed successfully and the product was tested at NCSU and performed well, as given elsewhere in this report.
Another option for production of this “voided slab” product is to eliminate the bottom flange which would allow the concrete to be placed in one stage without fear of having air voids in a 1-inch thin flange. The forms are still more complicated that a double tee form due to the fact that bottom bulbs as shown in Figure 11.4-4 would be required to house the bottom strands and to provide adequate member stiffness for vibration analysis. The option also allows for easier placement of utilities before a ceiling board is installed. This option will be fully explored in Phase II. Preliminary analysis in Phase I indicates cost effectiveness and even possible reduction in number of strands required for a 60 ft span. The equivalent solid slab thickness with this option is about 3.2 inches which shows high efficiency.

Figure 11.4-4. Option for One Stage Casting to be Explored in Phase II

11.5 Development Box Beams for Bridges

Box Beams have been a popular product for bridges in the 50 to 120 span range, especially in the Northeast region of the US where population is dense and available headroom is limited. This system offers the best combination of speed of construction, due to elimination of need for forming and placing CIP decks, and shallow depth due to the box’s high inertia/depth ratio. Interestingly, this popular system is also suitable for UHPC application. It can be shown by analysis that top and bottom flanges can be reduced to 4 in., enough to allow one row of strands. It can also be shown that a web width of 2 in. can be used for the two webs of the box, see Figure 11.5-1. When the beams are set adjacent to each other, a gap is required to create a moment and shear resistance longitudinal joint. As shown in the figure, the shape of the longitudinal joint is such that a shear key is created. Using UHPC for the field cast joint, would also allow the 6-inch gap at the top and bottom to have fully developed #6 bar lap splice as demonstrated by Graybeal (Need ref.). This detail would allow for full transverse continuity without aid of transverse post-tensioning or heavy intermediate diaphragms. With the proposed dimensions, as much as 60 percent of the concrete and most of the rebar can be saved. This would allow for wider beams than the standard AASHTO 3-ft and 4-ft wide boxes. In turn, the number of longitudinal joints is reduced and construction is accelerated.

A challenge of all voided products, including the box beam, is placement and securing of the one-time use EPS forms while concrete is being placed. Another challenge is the ability to ensure that the bottom flange concrete is filled...
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with uniform and consistent UHPC material without fear of air voids or fiber interruption by the bottom strands. Both challenges can be eliminated by casting the bottom half of the beam upside-down as recommended for the building voided slab. Such process, while it makes good sense theoretically, may receive strong resistance from the precasters and owners. As indicated earlier, fluted UHPC-UHPC interface has much larger ability to resist horizontal shear than roughened conventional concrete interface. This issue will be further verified, and a two-part system, will be demonstrated experimentally in Phase II, including consideration of required tolerances and differential camber.

![Figure 11.5-1](image)

**Figure 11.5-1. Possible Box Beam for Bridge Applications Requiring Shallow Structural Depth**

11.6 Development of Decked I Beam for Bridge Spans up to 250 ft

The most commonly used precast prestressed concrete bridge beam shape in the US is the I-beam. It has been shown over the years to be the most economical systems for spans in the range of 80-200 feet. Tools are available for design including avoidance of instability of long slender beams due to handling and shipping. It is difficult to justify using UHPC for I-beams such as the popular NU I-beam, California Wide flange and Florida I beam (FIB). Such beams are already highly optimized and the goal of saving 50 percent of the concrete volume is hard to achieve. However, one can still preserve the I beam shape and achieve significant economy by combining the beam with the deck in one precast unit, called decked I-beam, see **Figure 11.6-1**. Savings are realized in eliminating duplication of the function of the beam top flange and the deck as a final compression resistant component of the beam in the longitudinal direction. A significant amount of reduction in concrete volume can be achieved by creating ribs in the top flange in the transverse direction as long as the top skin of the ribbed slab is capable of resisting punching shear, which UHPC is excellent in achieving. Thus, a skin of 2 in. has been shown by analysis in this Phase of the Project to be adequate. This will be confirmed in Phase II. The total depth of the ribbed slab is preserved as the standard conventional deck depth. This will allow for preserving stiffness of the transverse direction.

![Figure 11.6-1](image)

**Figure 11.6-1. Decked I-Beam Proposed for use with UHPC for a 250-ft Span**

The 4-inch web width is shown by calculations based on existing state of the art to be adequate to resist vertical shear without exceeding maximum shear limits and without requiring stirrups. Thus, the section only has the UHPC, strands and bars in the stems of the top flange for transverse loading. Perhaps, a small amount of bursting steel rebar is needed at the beam ends to control horizontal web cracking. This was determined in the design example
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presented in this report to be one leg #6 at 3 in spacing which is less than that required with conventional reinforcement as the UHPC has significant tensile strength that could be mobilized to contribute to the rebar capacity to control bursting cracks.

Use of 0.7-inch strands is very favorable in the section shown in Figure 11.6-1. It has been demonstrated in the example that 54-0.7 in. strands are enough to resist loading on a 250 ft span with 9 ft total depth and 9 ft beam spacing. Also, use of Grade 100 rebar allows for the ribs to be spaced at 24 inch while only one bar is needed at the bottom of each rib, as shown in the calculations in the Appendix to this report.

11.7 Additional Potential Products

Deck panels, whether partial depth or full depth, can be optimized for use of UHPC. Figure 11.7-1 shows a possible solution for partial depth stay-in-place (SIP) deck panel that would be supported on the flange edges of a precast concrete beam. It has a total average thickness of 1.5” with ½” corrugations to enhance composite action with CIP conventional deck topping, about 5 ½ in. thick. The panel is pretensioned in the transverse direction with 3/8” strands at 9 in. spacing, which can be considered part of the transverse deck reinforcement for positive moment. The main value of this option is a reduction in the total typical thickness of 3 ½ to 4 in. Also, there is excellent protection of the deck from saltwater spray.

Figure 11.7-1. Partial Depth Deck Panel Produced by Concrete Technology Corporation as Part of Their Trial Mixing

Another deck product is being contemplated for Phase II testing. It is similar to that in Figure 11.7-1 except that the panel covers multiple bays, up to full bridge width. When this is done, it offers the opportunity to avoid the difficult to form overhang part of the deck. For reinforcement of the “full width” panel, it is proposed to use a steel truss similar to that currently used in Spain and other European countries. The truss must be capable of resisting the negative moment created by the weight of the wet concrete topping and the machines that helps in its placement.

Another, possibly cost-effective application is tall concrete poles for the power transmission industry, which is dominated by steel poles, and by spun-cast concrete poles at the lower end of the height range. Figure 11.7-2 shows a top view of a 120 ft tall pole with the width varying from 36 in. at the bottom to 12 in. at the top. The thickness is uniform at 2.5”. The pole is prestressed with 12-0.7 in. strands that are reduced to 6 strands at the top. The strand terminations are possible as the pole is made of two identical halves that are epoxied and bolted together. This patent pending system is not part of the current program and is just offered here to give an illustration of the huge potential impact on all facets of the precast prestressed concrete industry.
The same can be said of use of UHPC in precast pretensioned piling. An attempt was made by Standard Concrete Products, with assistance from e.construct, to optimize the 24 inch square pile. The pile was converted to an octagonal shape with a circular void, such that the wall thickness is reduced to only 4 inches. Pile segments were made and tested by FDOT with success. The pile had matched the flexural capacity of the conventional concrete pile and far exceeded the axial capacity which is critical in pile driving.

11.8 Short Term Implementation Strategy for I-Beam Bridges

One of the impediments to rapid implementation of UHPC in construction projects involving precast prestressed products is the costs associated with acquisition of new steel forms. Yet, full advantage of the cost-effectiveness of UHPC cannot be realized without consideration of a total systems approach. For example, the fact that one can reduce the total volume of concrete by nearly 50 percent with use of UHPC, allows for the more structurally efficient decked I-beam to be used for spans in the 150 to 200 ft range without exceeding most transportation and handling equipment capacity. It will allow for accelerated bridge construction since the deck is already integrated as the top flange of the beam. The 2-inch top flange skin is adequate for resistance of the wheel loads in punching shear mode and it significantly reduces the weight compared to a solid 8-inch thick conventional concrete slab.

Implementation of decked I beam system with a ribbed slab top flange requires more difficult formwork than that needed for a traditional I-beam. Although, forming cost is a one-time initial capital investment, private precasters would hesitate to make such investment without assurance of continuing use of the forms.

As a short-term strategy for this challenge, one can make the decked I-beam in two stage casting. The first stage would be production of an optimized I-beam based on the available forms, for example the standard NU (Nebraska University) I-beam shape see Figure 11.8-1. The new shape is created by moving the side forms closer to each other to create the required web thickness of say 3-4 inches. The bottom of the bottom flange is blocked out with 2-inch thick plastic (or steel) block. The top flange overhangs are reduced to a width of about 12-24 inches.
Once the first stage concrete is produced, second stage ribbed slab can be placed on top of it and the two connected with haunch and deck block-out UHPC, see Figure 11.8-2(c). This process takes longer to produce each decked I-beam than one-stage production. It is also less efficient structurally, as can be seen from the number of strands required in the example given in Table 11.8-1. However, it does have some benefits. It is possible to adjust for cross slopes to create a roadway crown. It is also to overcome undesirable camber and adjust the top face for a more suitable roadway profile.

As indicated earlier, the high unit cost of UHPC is offset with reduced fabrication labor due to elimination of most of the reinforcing bars and reduced material volume and weight. A target reduction of about 40-50 percent of concrete volume, as shown in the table, would likely produce UHPC systems that are cost competitive on initial cost basis, in addition to the numerous other benefits UHPC offers.
Figure 11.8-2. Cross-Sections for a 110-ft Span Bridge, (a) Conventional NU1100 with Conventional Composite Deck; (b) Proposed UHPC Decked I-Beam; (c) Possible Short-Term Implementation using Two-Stage UHPC Decked I-Beam
Table 11.8-1. Comparison Between Conventional Concrete and Two UHPC Options

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<th>Conventional NU 1100</th>
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<th>Percent reduction due to use of UHPC</th>
<th>Two-Stage UHPC, Modified NU100+ribbed slab</th>
<th>Percent reduction due to use of UHPC</th>
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<td></td>
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</tbody>
</table>

* Example: span = 110’-0”; spacing = 8’-6”; six girder lines

**Quantities are given per girder line

11.9 References


A. APPEndIX A - DRAFT GUIDELINES FOR PRODUCTION

A.1. Introduction to UHPC

Ultra-high performance concrete (UHPC) is a high-strength, high-durability composite material consisting of a cementitious matrix with an ultra-low water-to-binder ratio (w/b) and high-strength steel fiber reinforcement. UHPC, as defined in this project, is typically self-consolidating, with a w/b less than 0.20, and is characterized by a minimum compressive strength of 18,000 psi (124 MPa), a minimum first-cracking flexural strength of 1,500 psi (10.3 MPa), a minimum post-cracking flexural strength of 2,000 psi (13.8 MPa), and strain-hardening behavior under tensile loading. These superior properties of UHPC are derived from the ultra-low w/b of the mix and optimized particle packing of the constituent materials, which typically include cement, silica fume, sand, and a fine supplemental material such as limestone powder, fly ash, ground silica, or slag cement. Tensile performance is enhanced through the incorporation of high-strength steel fibers at a typical content of 2 percent by volume.

This document presents an overview of UHPC production specific to long-span precast, pretensioned structural elements. Topics discussed include the materials used to develop mix designs, batching and placement considerations for production, and methods for evaluating the performance of UHPC materials for mixture qualification and routine quality assurance. While many of the practices discussed in these guidelines are applicable to the production of a variety of UHPC elements, the specific focus of this document is the production of long-span precast, pretensioned structural elements for buildings and bridges, designed according to the Guidelines for Structural Design with materials that comply with the UHPC Materials Guide Specification prepared as part of this project.

A.1.1. Background

Ultra-high performance concrete was developed in the 1990s as a natural progression in improved performance of cementitious composites following the development and introduction of highly efficient high-range water reducers (HRWRs) (Naaman and Wille, 2012). Using these admixtures, water-binder ratios could be reduced to levels below 0.20, while significant quantities of fine and ultra-fine materials such as silica fume and silica flour could be incorporated into a mixture without sacrificing workability - both leading to increases in strength and microstructure density. The incorporation of high-strength steel fiber reinforcement has enabled UHPC to achieve significant tensile strength and post-cracking ductility in addition to high compressive strength and excellent durability properties.

UHPC production in North America has been most commonly based on the pre-packaged, proprietary materials, reportedly providing compressive strengths of up to 30,000 psi (206 MPa), uniaxial tensile strengths in excess of 1,000 psi (6.9 MPa), and a maximum uniaxial tensile elongation of more than 0.1% before failure (Chanvillard and Rigaud, 2003). Other proprietary UHPC materials are available worldwide, with UHPC prompting interest in France, Switzerland, Canada, Malaysia, and Japan, and additional development in Central and South America, Australia, and Southeast Asia.

Because of the potential benefits, a number of researchers, producers, and transportation agencies have begun to develop alternative, non-proprietary UHPC mixtures based primarily on locally-available materials. In 2010, Wille et al., developed a non-proprietary UHPC mixture with compressive strengths of 22,000 psi, uniaxial tensile strength of 800 psi, and a maximum uniaxial tensile ductility of 0.6% (Wille and Boisvert-Cotulio, 2013). They later modified this mixture to achieve a compressive strength in excess of 30,000 psi, uniaxial tensile strength of 5,400 psi and a maximum tensile ductility of 1.1% (Wille et al., 2011). More recent efforts by state departments of transportation (DOTs) and individual precasters have focused on the development of UHPC mixtures based on materials already used by or readily available to precast or ready-mix suppliers. These efforts may result in UHPC mixtures with lower compressive strengths compared to proprietary materials, but with comparable tensile performance, as the tensile performance is most significantly influenced by the type and quantity of fibers. For example, recent research sponsored by the Montana DOT has led to the development of a local UHPC mixture with compressive strengths of 18,000 to 20,000 psi and an ultimate flexural tensile strength of 3,400 psi (Berry et al., 2017).
Despite the many technological advances achieved over the last two decades with this material, UHPC has only just begun to penetrate the structural precast concrete market. The purpose of this document is to provide guidance to precasters who wish to develop UHPC production capabilities, encouraging more widespread use of UHPC in structural precast applications.

A.1.2. Benefits of UHPC

UHPC offers unique benefits that may make it more desirable for certain structural applications compared to conventional concrete, high-performance concrete, or even structural steel. A primary benefit of UHPC for structural precast applications is that the material’s high compressive strength and tensile ductility enable elements to be designed with smaller cross-sections and less reinforcing steel, reducing the overall weight of the element and the quantity of material required. These lighter cross-sections can be used for elements with long-span length requirements, or to eliminate intermediate supports in structures that can be re-designed to take advantage of longer span capabilities. The lighter-weight elements may reduce costs associated with transportation and handling and may further reduce other construction-related costs.

An additional benefit of UHPC for structural precast applications is the enhanced material durability brought about by the ultra-low w/b, the high volume of supplementary cementitious materials, and the dense particle packing of the constituent materials. Many UHPC mixtures offer exceptional durability in aggressive environments, with a high resistance to chloride ingress, sulfate attack, and other processes driven by moisture transport, such as cyclic freezing and thawing. These attributes make UHPC an attractive alternative to conventional or high-performance concrete for structures requiring long service lives or for structures exposed to highly aggressive environments.

Another important benefit of UHPC for structural precast applications is the potential to reduce the environmental impact of construction by developing designs that effectively utilize the enhanced properties of this material. When the compressive strength, tensile performance, and durability of UHPC are fully utilized to design elements with reduced cross-sectional areas and less reinforcing steel, in structures with fewer or lighter intermediate support structures, the overall volume of energy-intensive materials such as cement and steel may be reduced. As a result, the environmental impact of the initial construction is also reduced. Further reductions in impact are possible for UHPC mixtures that contain large volumes of supplementary cementitious materials, such as fly ash or slag, which can decrease the energy consumption and greenhouse gas emissions associated with cement usage. Over time, further environmental benefits of structures made with UHPC may continue to be realized due to the superior long-term durability offered by the UHPC, which reduces the need for maintenance, repair, rehabilitation, and re-construction activities. In this way, UHPC can present a lower environmental impact alternative to conventional construction materials.

A.1.3. Challenges of UHPC

UHPC is not a solution to every problem, and may present a number of challenges that make it less desirable for certain applications. One of the primary challenges to implementation of UHPC in structural applications, in general, is the comparatively higher unit material cost and the perception that the material is “too expensive” to be a viable solution for an owner. However, costs may be offset by using locally-available materials, using efficient designs that reduce the total amount of concrete and reinforcing steel required, considering the reduced transportation charges, and considering the reduced the need for long-term maintenance and repairs.

Another challenge to implementation of UHPC in structural applications is the increased complexity of material production and placement relative to conventional concrete mixtures. However, precasters have been able to successfully adapt and produce precast UHPC products. Many of the complexities associated with UHPC may be overcome by following the recommendations and guidance presented in this document.

A.1.4. Objectives of these Guidelines

The primary objective of this document is to provide a practical guide for the development and qualification of UHPC mixtures based on locally-available materials, and for utilization of those UHPC mixtures for the production of long-span precast, pretensioned concrete elements. Guidance provided in this document includes:

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Appendix A – Draft Guidelines for Production

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A.1.5. References
These guidelines for production were developed as a concise synthesis of the project team’s UHPC production experience and the currently available information regarding UHPC production and implementation. Development of these guidelines relied, in part, on the information contained in the following documents. A complete list of references for these guidelines is provided in Chapter 7. (Additional references are given in the Phase 1 report.)

American Concrete Institute (ACI)
- ACI 239RX, Guide on Materials and Methods of Construction for UHPC (draft)

ASTM International

Canadian Standards Association (CSA)
- CSA A23.1 Annex U (Informative), Ultra-High Performance Concrete (UHPC) (2019)

Federal Highway Administration (FHWA)

German Committee for Structural Concrete (DAfStb)
- DAfStb-Guideline for Ultra-High Performance Concrete, 2017

Korea Concrete Institute (KCI)
- KCI-M-12-003, Guideline for K-UHPC Structural Design (2012)

French Standardization Association (AFNOR)
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National Precast Concrete Association (NPCA)


Swiss Society of Engineers and Architects (SIA)


Japan Society of Civil Engineers (JSCE)

A.2. Definitions of UHPC

Ultra-high-performance concrete (UHPC) is typically distinguished from other types of concrete based on its mechanical performance. When defining a concrete material as “UHPC”, the most common defining feature is the minimum compressive strength of the material. Nearly every international UHPC standard, guide document, and state or federal agency publication includes a minimum compressive strength that distinguishes “UHPC” from other types of high-performance concrete mixtures. Typically, this minimum required compressive strength ranges between 17,000 psi and 22,000 psi. While compressive strength is the most readily measured property, it is not the most important characteristic of UHPC. Accordingly, many definitions of “UHPC” also include a minimum tensile strength and/or ductility requirement, and many also specify minimum durability criteria. A summary of UHPC definitions sampled from prominent references in the UHPC literature is presented in Table A.2.1.

Table A.2-1. Minimum Definitions of UHPC

<table>
<thead>
<tr>
<th>Document</th>
<th>Country</th>
<th>Minimum Compressive Strength (psi)</th>
<th>Minimum Tensile Strength* (psi)</th>
<th>Other Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 239R-18</td>
<td>United States</td>
<td>22,000</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>AFNOR NF P 18-470</td>
<td>France</td>
<td>18,800</td>
<td>870, first crack</td>
<td>Durability, ductility, and fire resistance</td>
</tr>
<tr>
<td>ASTM C1856-17</td>
<td>United States</td>
<td>17,000</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>CSA A23.1, Annex U</td>
<td>Canada</td>
<td>17,400</td>
<td>580, first crack</td>
<td>Durability and ductility</td>
</tr>
<tr>
<td>FHWA, various documents</td>
<td>United States</td>
<td>21,700</td>
<td>720, post-crack</td>
<td>w/cm ≤ 0.25</td>
</tr>
<tr>
<td>SIA 2052</td>
<td>Switzerland</td>
<td>17,400</td>
<td>1000, first crack</td>
<td>Ductility</td>
</tr>
</tbody>
</table>

* Tensile strength is specified by either direct, uniaxial tensile testing or by indirect inverse analysis of flexural testing results.

While many definitions for UHPC currently exist worldwide, for the purposes of these Guidelines and for use in the production of structural precast pretensioned concrete products designed according to the design guidelines, the following specific definition has been adopted: UHPC is a self-consolidating, fiber-reinforced cementitious material with the minimum physical properties defined in Table A.2.2.
## Table A.2-2. Minimum Properties for Precast, Prestressed UHPC

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Minimum Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow spread</td>
<td>ASTM C1856</td>
<td>8 to 11 inches, measured not longer than 15 minutes before placement</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>ASTM C1856</td>
<td>≥ 10 ksi at prestress release&lt;br&gt;≥ 18 ksi at service</td>
</tr>
<tr>
<td>First-peak (first crack) flexural strength, $f_{fc}$</td>
<td>ASTM C1856</td>
<td>≥ 1.5 ksi at service</td>
</tr>
<tr>
<td>Peak (ultimate) flexural strength, $f_{fu}$</td>
<td>ASTM C1856</td>
<td>≥ 2.0 ksi at service</td>
</tr>
<tr>
<td>Ratio of Peak (ultimate) flexural strength $f_{fu}$ to First-peak (first crack) flexural strength, $f_{fc}$</td>
<td>ASTM C1856</td>
<td>≥ 1.25 at service</td>
</tr>
<tr>
<td>Residual flexural strength</td>
<td>ASTM C1856</td>
<td>At midspan deflection of L/300: ≥ 90 percent of first-peak (first-crack) strength at service&lt;br&gt;At midspan deflection of L/150: ≥ 75 percent of first-peak (first-crack) strength at service</td>
</tr>
<tr>
<td>Resistance to Chloride Ion Penetration</td>
<td>ASTM C1856, performed on specimens without steel fibers</td>
<td>≤ 500 coulombs at 28 days (for structures exposed to chlorides)</td>
</tr>
</tbody>
</table>
A.3. UHPC Materials and Mixture Proportioning

The enhanced mechanical properties and durability of UHPC are largely derived from a dense microstructure created through optimized packing of the constituent materials. Proper selection and proportioning of the constituent materials is critical to creating the dense microstructure that imparts the exceptional material characteristics to the UHPC composite. This chapter discusses materials that are conventionally used in UHPC and procedures for selecting and optimizing the material proportions. Methods for qualifying and characterizing UHPC mixtures are described in Chapter 2.

A.3.1. Basis for UHPC Mixture Development

The desired performance characteristics of UHPC are achieved largely through dense packing of the constituent materials, combined with an ultra-low water-binder ratio (w/b). When the materials are properly graded and tightly packed, the average size of the pores within the concrete is reduced, as illustrated schematically in Figure A.3.1-1. The reduced size and volume of capillary porosity significantly aids in increasing compressive strength and improving long-term durability, similar to how dense particle packing improves the mechanical performance and durability of conventional and high-performance concretes containing silica fume (Neville, 2011).

The dense packing is achieved through selection of appropriately sized constituent materials that cover a wide range of particle sizes, from aggregates at the coarsest end of the range to silica fume at the finest. The materials are proportioned in such a way that an optimized packing gradation may be achieved.

When materials are optimally graded, not only is the capillary porosity reduced and the resulting strength and durability improved, but the water demand of the plastic UHPC is also reduced. In effect, the tightly-packed particles fill the space that would have otherwise contained mixing water, which frees up the mixing water to better coat and lubricate the raw materials (Wille and Boisvert-Cotulio, 2013). This allows UHPC to become highly workable and fluid, even at ultra-low w/b. However, because of the ultra-low w/b and the typically high fineness of the constituent materials, use of an efficient high-range water reducing admixture (HRWR) and significant mixing energy are necessary to effectively disperse the water and the raw materials during mixing. Accordingly, the types of materials and mixture proportioning used for UHPC typically have a significant impact on mixing time and efficiency when using conventional mixers. Considerations related to the constituent material and admixture selection are described in further detail in Section A.3.2.
While the compressive strength and durability of UHPC are enhanced through particle packing and an ultra-low w/b, the tensile performance is primarily achieved through effective distribution and development of fiber reinforcement. Fiber reinforcement provides the UHPC with increased tensile strength and post-cracking ductility, to the extent that the tensile capacity of the UHPC can be utilized in the design of structural elements. As described in further detail in Section A.3.2.1, the type, size, proportion, and anchor of the fibers all contribute to the tensile characteristics of the UHPC (Naaman, 2017; Balaguru and Shah, 1992), so selection of the fiber type(s) and content(s) is a key aspect of UHPC mixture design and development.

The tensile behavior of UHPC can be broadly categorized into three stages, as illustrated in Figure A.3.1-2. In the first stage, the UHPC exhibits linear-elastic behavior, with a linear stress-strain relationship defined by the tensile elastic modulus of the material, up until the concrete first cracks. In the second stage, the UHPC undergoes multiple cracking while the tensile capacity continues to increase to its peak (strain-hardening). In this stage, a portion of the tensile load is carried by the fibers, which are still anchored into the concrete and therefore provide tensile restraint across the cracks. The fiber bridging allows multiple cracks to form in the UHPC without sudden failure, and is characterized by the saw-tooth pattern portion of the stress-strain curve. In the third stage, the tensile stresses begin to localize at a single crack location, and the material exhibits strain-softening behavior as the fibers along the crack lose their bond with the concrete and pull out of the matrix under continued loading. Because not all of the fibers pull out at once, the material is able to sustain tensile loads at a large strain, providing UHPC with its characteristic high ductility.

**Figure A.3.1-2. Schematic Illustration of a Tensile Stress versus Strain Diagram for UHPC. The Tensile Performance is Characterized by Three Stages: (I) Linear Elastic Behavior until First Crack (at First-Peak Strength), (II) Multiple Cracking and Strain Hardening until the Peak (Ultimate) Strength is Reached, and (III) Strain Softening as Stresses Localize at a Single Crack Location. The Total Energy Absorbed by the UHPC is Indicated by the Shaded Area under the Curve (Figure Based on Russell and Graybeal, 2013).**

A UHPC mixture is generally proportioned to achieve a balance of workability in the fresh state and strength and durability in the hardened states. Achieving the target performance often requires tailoring the mixture proportions and mixing procedures to the specific mixer used for production (Figure A.3.1-3). Although many producers develop preliminary UHPC mixtures in the laboratory, the final mixture proportions must be determined based on actual production-scale batching, using the plant’s own equipment. For example, certain
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high-range water reducers (HRWR) may exhibit different wetting efficiencies in a production-scale pan mixer than in a laboratory-scale high-shear mixer. These differences may only be identified through production-scale trial batching. Therefore, while the guidance presented in this section largely considers mixture proportioning based on laboratory trials, additional adjustments to the final mix design, including the constituent materials and their relative proportions, may be required based on subsequent production-scale trials and robustness evaluations (see Section A.4.2).

![Diagram](image)

*Figure A.3.1-3. Achieving Satisfactory Mixture Performance Requires a Balance in the Mix Design, the Mixer, and the Mixing Procedures.*

A.3.2. Mixture Development

A.3.2.1. Constituent Materials

UHPC generally consists of a cementitious matrix, fine aggregates, and fibers. The types of raw materials used in the production of UHPC may be the same or similar to those used for conventional concrete, but material quantities and properties may vary significantly. A typical cementitious matrix for UHPC contains portland cement, silica fume, and one or more supplemental materials that are blended to create a combined particle size distribution with optimal packing; however, successful UHPC mixtures have also been developed that contain no silica fume (Khayat and Valipour, 2018; Rougeau and Borys, 2004) or only silica fume without supplemental materials (Rossi, Arca, Parant, and Fakhri, 2005; Russell and Graybeal, 2013; Alsalmán, Dang, and Hale, 2017). Selection of constituent materials for UHPC should consider the materials’ chemical properties and reactivity, as well as their physical properties, including particle size and dispersion quality, and the material uniformity and consistency.

A.3.2.1.1. Cement

Various types of portland cement have been used for the production of UHPC. To date, the most commonly used cements used in UHPC have been Type I and Type I/II cements due to their widespread availability, but Type III and Type V cements have also been used successfully (Wille and Boisvert-Cotulio, 2013; Russell and Graybeal, 2013). Specialty cements including Class H oil-well cements and white cements have also been used (Wille and Boisvert-Cotulio, 2013; Scott, et al., 2015). These specialty materials tend to have lower C3A contents and slower rates of hydration compared to ordinary portland cements, and may therefore impart more favorable workability characteristics to the fresh UHPC.
Cements used for the production of UHPC should conform to the requirements of the relevant ASTM, AASHTO or other governing material specifications (e.g., ASTM C150, AASHTO M85, American Petroleum Institute [API] SPEC 10A). The primary characteristics to consider when selecting a cement for UHPC are the cement's C₃A content and Blaine fineness. To reduce workability challenges arising from rapid rates of hydration and to facilitate better admixture compatibility (Neville, 2011), it is recommended that the cement have a C₃A content no greater than 8 percent and a low to moderate Blaine fineness, less than 400 m²/kg. Cements with low or moderate heats of hydration (less than 70 kcal/kg [290 kJ/kg] at 7 days) also tend to provide more favorable workability due to their typically slower rates of hydration. Cements not possessing these chemical and physical properties may still be used in UHPC; however, workability, particularly long-term flow retention, may become a challenge if the properties of the cements deviate significantly from these target values.

### A.3.2.1.2. Silica Fume
Silica fume is commonly used in UHPC because of its fine particle size, which promotes dense particle packing (Figure A.3.1-1), and its high pozzolanic reactivity, which provides early strength to the hardened UHPC. In conventional and high-performance concretes, silica fume is typically limited to no more than 5 to 10 percent of the total cementitious material due to workability challenges seen at higher levels (e.g., sticky consistency, high viscosity, difficulty finishing, and increased tendency for plastic cracking); however, in UHPC, silica fume could be as high as 30 percent by weight of cement, as it increases the density of the cementitious matrix (Russell and Graybeal, 2013). This high dosage of silica fume, combined with the typically low water-cement ratios specified for UHPC mixtures, necessitates increased dosages of high-range water reducer to achieve sufficient workability (see Section A.3.2.1.5).

Silica fume used for UHPC production should conform to ASTM C1240. Agglomerated or densified silica fume may be used effectively, but consideration should be given to mixing the agglomerated silica fume with the fine aggregates prior to addition of the remaining powder materials to improve dispersion of the silica fume particles throughout the cementitious matrix (see Chapter 4). Silica fumes with high silica contents (SiO₂ > 95 percent) are preferred for pozzolanic activity, and selection of a silica fume with as low a carbon content as possible is recommended for improved compatibility with chemical admixtures.

### A.3.2.1.3. Supplemental Materials
Other “supplemental materials” are also conventionally used in UHPC. These materials may include supplementary cementitious materials (SCMs), such as fly ash, slag cement, or metakaolin, or mineral fillers, such as ground quartz (silica flour) or limestone powder. The primary purpose of the supplemental materials in UHPC is to optimize the packing density of the material by providing an intermediate particle size for a more continuous dense gradation. Improved particle packing through the use of supplemental materials has been associated with improvements in workability, mechanical strength, and durability (Wang, et al., 2015). It has been found that higher strengths may be achieved with a non-reactive mineral filler with a more optimal gradation than may be achieved with a similar proportion of a reactive SCM with a less ideal gradation (Wille and Boisvert-Cotulio, 2013). It is recommended that supplemental materials used in UHPC have a particle size between that of cement and silica fume, with a target nominal maximum particle size less than 0.003 inches (80 µm) (Canadian Standards Association, 2019); however, supplemental materials having particle sizes between cement and fine aggregate may also prove beneficial.

Supplemental materials used in UHPC should conform to the requirements of applicable ASTM or other governing specifications. Fly ash and metakaolin should conform to ASTM C618, slag cement should conform to ASTM C989, and limestone powder should conform to ASTM C1797. In general, good mechanical performance has been achieved for UHPCs utilizing supplemental materials with well-controlled and consistent chemistries and having clearly defined and consistent particle size distributions (e.g., silica flour) (Wille and Boisvert-Cotulio, 2013). Materials with more variable compositions or particle sizes may lead to increased variability in the fresh and hardened properties of the UHPC.

### A.3.2.1.4. Mixing Water
Water used for UHPC production should conform to the same standards as water used for conventional concrete production. To control temperatures during production and placement (see Chapter 2), it may be
desirable to use chilled water or to substitute a portion of the mixing water with ice. In general, cubed ice is considered more effective than chilled water at controlling UHPC temperatures due to its lower temperature. In addition, the increased shearing action provided by the ice can aid in breaking up the powder materials (Graybeal, 2014).

If ice is used, it should be substituted for water on a 1:1 basis by weight. The ice should be of a small enough size to completely melt during mixing and proportioned such that sufficient water is available for the mixture to turn fluid during mixing. Mixing and uniformity challenges may arise if a large percentage of the mixing water is replaced by ice (Precast/Prestressed Concrete Institute, 1999). All ice should be completely melted prior to addition of the fiber reinforcement (Graybeal, 2014).

A.3.2.1.5. Chemical Admixtures

Chemical admixtures are an essential component for the production of UHPC. Chemical admixtures used in UHPC should conform to all applicable requirements of ASTM C494, ASTM C1017, and ASTM C260, as appropriate, and should not have a deleterious impact on the performance of the hardened UHPC. The water content of all admixtures must be considered in calculating the total water content of the UHPC.

- **High-range water reducers (superplasticizers)** are necessary to provide enough workability to mix and place UHPC at the low water-cement ratios used for this material. These admixtures work by separating the powder materials from one another during mixing, which frees up mixing water to coat and lubricate the particles (Neville, 2011). High-range water reducers (HRWR) used for UHPC must be effective for mixtures with high powder contents and must not adversely affect the hardened properties of the UHPC when used at high dosages (which may be above the manufacturer's recommended levels for conventional concrete). Admixtures based on polycarboxylate technology are recommended, because of their high efficiency in dispersing powder materials. HRWRs for conventional concrete may be used for UHPC, but dosages much greater than the manufacturer's recommendations are often required. In UHPC mixtures, a "saturation" phenomenon may be observed, in which increasing dosages of HRWR beyond a certain level have no additional beneficial effect on workability (e.g. flow) (Neville, 2011). If, for a combination of materials optimized for particle packing, saturation of the HRWR occurs below the workability target, the only way to increase workability may be to increase water content. However, increasing water content to improve workability may have the negative effect of reducing strength and durability. Water-reducing admixtures specifically tailored to the unique needs of UHPC mixtures are currently available and are anticipated to become more widely available as the use of UHPC becomes more widespread. These admixtures can generally be used at lower dosages than conventional high-range water reducers, making them less susceptible to saturation effects.

- **Set-retarding or accelerating admixtures** may be used to control the rate of hydration or setting behavior of the UHPC. Set-retarding admixtures may be desirable if extended working times are required, while accelerating admixtures may be desirable if earlier strength gain is necessary (e.g., for earlier release of prestressing) or if shorter setting times are desired. It is worth noting that many HRWRs have set-retarding characteristics - especially when used at the high dosage levels common in UHPC. Accordingly, UHPC mixtures may have long setting times (e.g. approaching 24 hours), even without the use of set-retarding admixtures (ACI Committee 239, 2018). As such, the set-retarding effect of HRWRs should be properly assessed, and set-retarding admixtures should be used with caution in cold weather, as extended setting times may result.

- **Air-entraining admixtures** are not commonly used for UHPC, as the low water-cement ratio and disconnected pore structure of the material make it highly resistant to deterioration from freezing and thawing cycles (see Section A.4.1). In addition, it may be exceptionally difficult to entrain an adequate air void system at very low water-cement ratios, especially for mixtures containing large volumes of silica fume and lacking coarse aggregates, as are typical for UHPC (Neville, 2011).
• **Other chemical admixtures** may also be used to impart particular characteristics to the UHPC. Workability retaining admixtures may be used to extend the working time of a plastic UHPC mixture without altering its setting behavior. Viscosity or rheology modifying admixtures may be utilized to alter the flow characteristics of the UHPC, to improve the stability of the fibers within the flowable matrix, and/or to improve the finishability of the plastic UHPC surface. Shrinkage reducing admixtures (SRAs) or expansive agents based on periclase (MgO) or lime (CaO) may be utilized to reduce or offset the autogenous shrinkage often associated with high cementitious content and low w/c concrete mixtures, which may be of particular concern for precast, prestressed UHPC elements (see Section A.4.1 and the design guidelines).

Similar to conventional concrete, incompatibilities between the chemical admixture(s) and the other mixture components, or between individual chemical admixtures may result in unintended negative consequences related to the performance of the UHPC in the plastic and hardened states. It is recommended that selection of chemical admixtures be performed in consultation with the admixture supplier to ensure that the admixtures selected are compatible with one another and do not result in detrimental performance, especially considering the high admixture dosage levels that may be necessary for UHPC.

**A.3.2.1.6. Aggregates**

Aggregates constitute an essential component of the UHPC matrix. The size, shape, surface texture, hardness, and porosity of the aggregates all influence the performance of the UHPC composite in both the fresh and hardened states.

Aggregates used for UHPC production should meet the aggregate quality requirements of ASTM C33 or ASTM C144, with the exception of gradation. Fine aggregates for UHPC are most commonly silica sands, but may also consist of silicon carbide sand, bauxite sand, or others (Wille and Boisvert-Cotu, 2013; Canadian Standards Association, 2019). To provide good workability, aggregates should be clean and well-graded, with low absorption to moisture.

The aggregates used for UHPC should be of a size that does not inhibit dispersion of the fibers or adversely affect strength. To effectively disperse the fibers through the matrix and to minimize the weak interfacial zone around the aggregates which may otherwise control tensile performance, the maximum aggregate size should be no more than one-third the length of the fiber (Naaman, 2017; Canadian Standards Association, 2019). Most commonly, the aggregates used in UHPC are fine aggregates, with a maximum size less than that of conventional concrete sand. Often, the maximum aggregate size for UHPC does not exceed 0.03 inches (0.8 mm, or passing the No. 30 sieve).

**A.3.2.1.7. Fibers**

Fibers used for UHPC production fall into two distinct categories: metallic fibers and non-metallic fibers. Metallic fibers, most commonly steel, are primarily used to enhance the post-cracking strength and ductility of structural UHPC elements. Non-metallic fibers, including glass and polyvinyl alcohol (PVA) fibers, may also be used to provide enhanced tensile performance (Naaman, 2017; Balaguru and Shah, 1992). Polypropylene (PP) and PVA fibers can also be used to improve fire resistance of both structural and non-structural UHPC elements (Naaman, 2017; ACI Committee 544, 2009).

Steel fibers are the most commonly used fiber type in UHPC. Steel fibers are available in a variety of sizes, aspect ratios, anchorage types, and tensile strengths. Each of these properties of the fiber will influence the tensile performance of the UHPC composite, as previously described. Fibers may be straight or may be crimped, twisted, or hooked to increase fiber pull-out resistance. Each anchorage type, in combination with the fiber geometry and material strength, will result in different performance characteristics for the UHPC composite (ACI Committee 544, 2009). Some fiber geometries and anchorage types may be more or less challenging to disperse evenly in the fresh UHPC: short fibers or fibers with hooked ends may be more susceptible to clumping during batching, while larger fibers may hinder workability. Short, narrow fibers with high aspect ratios (> 60) and high tensile strengths (> 300 ksi [2100 MPa]) are preferable for more efficient dispersion through the fresh UHPC and for the resulting high tensile capacity of the hardened UHPC (Naaman, 2017). Steel fibers used in
UHPC should meet the requirements of ASTM A820; steel fibers used in transportation products may also need to meet Buy America requirements.

The fibers used in this project are brass coated steel fibers with 0.2 mm (0.008 in.) diameter and 13 mm (1/2 in.) or 20 mm (3/4 in.) length. The minimum tensile strength is 2500 MPa (362 ksi). This type of fibers was chosen for this project as it is the type that has been generally used in North America for UHPC by Lafarge and other commercial suppliers of pre-bagged UHPC.

Polymeric fibers may be used in structural precast applications as a supplement to steel fibers to provide non-structural characteristics, such as improved fire resistance, to the UHPC. Both PVA and PP fibers melt at elevated temperatures, providing channels for steam to enter and relieve internal pressures. Polymeric fibers are recommended at a minimum dosage of 0.2 to 0.3 percent by volume (Canadian Standards Association, 2019) for improved fire resistance; higher dosages may cause negative effects and additional challenges with respect to mixture workability. Certain types of polymeric fibers (e.g., PVA fibers) may also contribute to improvements in tensile strength.

A.3.2.2. Mixture Proportioning
Unlike conventional concrete, the performance of UHPC is highly dependent upon the chemistry and particle packing of the constituent materials, and therefore the proportioning of UHPC mixtures is integrally related to the unique chemical and physical attributes of the constituents. UHPC mixture proportioning is generally an iterative process, in which multiple trial mixtures are batched in the laboratory under a controlled environment. The laboratory trial batches are typically evaluated on the basis of plastic properties such as flow spread, with hardened properties such as strength usually evaluated only after a mixture has demonstrated satisfactory performance in the plastic state (see Chapter A.4 for discussion on performance evaluation). Once adequate performance has been demonstrated through laboratory trial batches, larger trials are then performed at the production scale, and further adjustments and refinements are made until the target performance can be demonstrated at a production scale.

A.3.2.2.1. General Properties of UHPC Mixtures
The most defining characteristic of a UHPC mixture is the ultra-low w/b that provides the hardened material with its exceptional compressive strength and durability. As shown in Figure A.3.2.1-1, most non-proprietary UHPC mixtures have a w/b of 0.20 or less, with higher strengths generally occurring at the lowest w/b’s. Note that while compressive strengths in excess of 25,000 psi (172 MPa) may be achieved at some ultra-low w/b’s, these mixtures do not always become “final mixes” used for field production due to other factors such as poor workability or high associated costs.
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Figure A.3.2.2.1-1. Compressive Strength of Non-Proprietary UHPC Mixtures versus w/b. Data Obtained from Mixture Development Work Performed for this Project and From the Following References: (Khayat and Valipour, 2018; Wille and Boisvert-Cotulio, 2013; Alsalman, Dang, and Hale, 2017; Berry, Snidarich, and Wood, 2017). Conversion note: 145 psi = 1 MPa

In addition to an ultra-low w/b, most UHPC mixtures also contain:

- 1500 to 2000 pounds per cubic yard (900 to 1200 kg per cubic meter) of total cementitious material;
- 10 to 30 percent silica fume, by weight of cement;
- 0 to 30 percent supplemental material, by weight of cement (Note: this may be significantly higher if slag cement is used or if multiple supplemental materials are used);
- One or more fine aggregates with a total weight 1 to 2 times the weight of cement; and
- 1 to 3 percent steel fibers, by volume (Note: UHPC elements designed according to the design guidelines must contain 2 percent steel fibers by volume).

While these ranges represent “typical” UHPC mixtures produced to date, UHPC mixtures have also been successfully implemented with different constituent materials, or with mixture proportions outside of these ranges.

A.3.2.2.2. Approaches for Mixture Proportion Development
The relative proportions of the constituent materials may be determined through:

- **Iterative laboratory trial batching**, in which successive trial batches are produced, changing one variable at a time, until the target performance characteristics are achieved. This approach is most similar to laboratory trial batching methods that may be used to determine mixture proportions for conventional or high-performance concretes (Kosmatka and Wilson, 2016). While this approach is conceptually the simplest, it may not be the most efficient means of determining mixture proportions and may not result in a mixture with the most “optimal” particle packing.
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- **Experimental design**, in which an experimental matrix is established to cover a wide range of potential mix proportions based on the available materials. All candidate mix proportions are batched and the resulting UHPCs are tested. Regression models are then applied to identify “optimum” proportions. This has the advantage over the iterative process in that multiple options are examined simultaneously, but it may be less efficient, as unfavorable combinations may be evaluated. For examples of experimental design applied to UHPC mix development, see Wille and Boisvert-Cotulio (2013) and Berry et al. (2017).

- **Particle packing models**, in which the relative proportions of the sand and powder constituents are selected to match as closely as possible a theoretical “optimal” packing of particles (based on material gradations). Multiple particle packing models have been used for UHPC mixture development, including those described by de Larrard and Sedran (1994), Fehling et al. (2014), and Yu, Spiesz and Brouwers (2015). While particle packing models can be used to quickly determine the relative proportions of the solid constituents, they are typically accompanied by iterative trial batching to determine the water content and admixture dosages necessary for achieving the target performance. Particle packing models are discussed in greater detail in the following section.

A.3.2.2.3. Mixture Proportioning by Particle Packing Model

A method found to be efficient during the execution of this project is to develop preliminary mixture proportions using particle packing models. While there are numerous particle packing models that may be utilized to determine the optimum gradation of particles, a common model used for UHPC material optimization that was also selected for this project is the modified Andreasen and Andersen model (Andreasen and Andersen, 1930). The modified Andreasen and Andersen model defines a dense distribution of particles according to the equation:

\[
P(D) = \frac{D^q - D^q_{\text{min}}}{D^q_{\text{max}} - D^q_{\text{min}}}
\]

where \(P\) is the percent of particles with a diameter smaller than \(D\); \(D_{\text{min}}\) and \(D_{\text{max}}\) are the minimum and maximum particle diameters, respectively; and \(q\) is a shape parameter between 0 and 1 (Brouwers and Radix, 2005; Funk and Dinger, 1994). As shown in Figure A.3.2.2.3-1, a \(q\) value closer to 1 produces a coarsely graded mixture, while a \(q\) value closer to 0 produces a finely graded mixture. For UHPC mixtures developed based on the modified Andreasen and Andersen curve, \(q\) is typically selected to be a value between 0.19 and 0.37, with values at the lower end of the range providing a finer mixture, and values at the higher end of the range providing a coarser (but still densely packed) mixture (Yu, Spiesz, and Brouwers, 2015; Wille and Boisvert-Cotulio, 2013; Yu, Spiesz, and Brouwers, 2014). A \(q\) value of 0.22 to 0.25 has been found to provide optimal characteristics for most UHPC mixtures (Yu, Spiesz, and Brouwers, 2014).
To develop a UHPC mixture using a particle packing model such as the modified Andreasen and Andersen model, the following steps are performed:

1. Review the available materials to assess suitability for use in UHPC (see Section A.3.2.1 for guidance). Depending on the quality and particle size distribution of the available materials, it may be beneficial to procure additional materials for the purpose of producing UHPC.

2. Determine the particle size distribution of each constituent material using particle size analysis techniques. A popular technique for cementitious materials and mineral fillers is laser diffractometry, with the cementitious materials and mineral fillers dispersed in a non-reactive liquid such as isopropanol. While particle size analysis of fine cementitious materials generally requires specialized equipment and analysis procedures, these techniques can be performed relatively inexpensively by many analytical testing labs. Additional information may also be readily available from some material suppliers.

3. Predict the “ideal” gradation for the materials using the selected particle packing model. In the case of the modified Andreasen and Andersen model, the $D_{\text{min}}$ and $D_{\text{max}}$ values are selected as the smallest and largest diameter particles present among the candidate materials. For other models, the “ideal” gradation may be determined by considering additional particle sizes.

4. Determine the “optimum” relative proportions of the candidate constituent materials by minimizing the difference between the combined gradation and the optimum gradation predicted by the particle packing model. The combined gradation for a proposed mixture design can be determined by taking a weighted average of the material gradations, with the “weights” equal to the percentage of each material included in the combination, or mathematically:

$$P_{\text{combined}}(D) = \sum w_i P_i(D)$$

where $P(D)$ is the percent of each material $i$ that passes through an opening of width $D$, and $w_i$ is the weight fraction of material $i$ in the mix. The relative proportions of materials are then varied (e.g. using a spreadsheet solver function) until the combined gradation with the “optimum” particle packing is obtained.
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5. Conduct laboratory trial batches to identify the appropriate water content and admixture dosages based on the relative proportions of solid materials determined by the particle packing model. The water content should be limited to produce a w/b of less than 0.21. These trials are typically performed on small batches without fibers, with iterative adjustments made to the mixing procedure, total water content, admixture types and dosages, and the “q” shape parameter (in an Andreasen and Andersen model) until suitable workability is achieved. Compressive strength testing may be performed on the material without fibers, either based on cubes or cylinders.

As discussed in Section A.3.2.1.5, at the high HRWR dosages typically used for UHPC, it may be observed that further increases in HRWR dosage do not result in further increases in flow spread and may actually cause a decrease in flow. If sufficient flow is achieved at this saturation level, additional testing is valuable to determine the lowest dosage of HRWR that achieves this level of flow. However, if insufficient flow (or just barely sufficient flow) is achieved at the saturation level, it may be necessary to consider an alternative HRWR, reevaluate the mixture constituents, or increase total water content. Adjustments to the constituent materials could include working with alternate cement or supplemental materials or setting limits on the proportions of a particular material, such as silica fume. If limits are established, the optimization process will need to be repeated.

Recommended mixing sequences for trial batching are discussed in Section A.5.2.2, and recommended performance targets for trial mixes are discussed in Section A.4.1. See Section A.3.2.3 below for a note on determining the w/b of the mixture.

6. Incorporate fibers into the mix and evaluate workability and compressive and tensile performance. This may be done based on laboratory trials or based on production trials (see next step.) Adjustments to the mix or mixing procedure may be made incrementally until sufficient performance is achieved.

7. Once sufficient performance is achieved in the laboratory, conduct additional trial batching at the production scale. Further adjustments are made to mixture sequencing, batching process and admixture dosages, as needed based on the available production facilities, until sufficient performance characteristics are achieved. If suitable performance cannot be achieved at the production scale, additional laboratory trial batches may be required to evaluate an alternative mixture design or mixture adjustment.

A.3.2.2.4. Example of Mixture Proportioning by Particle Packing Model
As a practical example, a mix designer would like to design a UHPC mixture consisting of cement, silica fume, and sand. The particle gradations for each material are shown in Figure A.3.2.2.4-1. The gradation of sand was provided by the material supplier, and the particle size distributions of cement was determined by a local analytical laboratory using laser diffractometry methods. The analytical laboratory did not have appropriate equipment to characterize the gradation of the silica fume, so a partial gradation was taken from a literature source as an “approximate” gradation for the silica fume (Elkom Materials, 2016).
The mix designer identifies the largest and smallest particle sizes in the system. In this case, the largest size, $D_{\text{max}}$, is assumed to be 1.2 mm (0.046 in), the maximum size aggregate, and the smallest size, $D_{\text{min}}$, is assumed to be 0.1 $\mu$m ($3.9 \times 10^{-6}$ in), the smallest size of silica fume that could be resolved by the selected particle size analysis technique. The designer decides to optimize the particle gradation based on the modified Andreasen and Andersen model, using a "q" shape parameter value of 0.25. The target, theoretical "optimum" gradation is shown in Figure A.3.2.4-2.
Figure A.3.2.4-2. Target Gradation by Andreasen and Andersen Model. (Note: 0.001 in = 25.4 µm)

The objective of the particle packing model optimization is to determine, for 1-part cement (by weight), what are the relative ratios of the other two materials that produce a combined gradation with the most “optimal” particle packing according to the selected model. The mix designer iteratively determines the combined gradation for a variety of potential proportions. For a mixture proportioned with 1-part cement : 0.25-parts silica fume : 1.2-parts sand (by weight), the weights, w, used to determine the combined gradation would be:

\[
\text{Total} = 1 \text{ (cement)} + 0.25 \text{ (silica fume)} + 1.2 \text{ (sand)} = 2.45
\]

\[
w_{\text{cement}} = \frac{1}{2.45} = 0.41
\]

\[
w_{\text{silica fume}} = \frac{0.25}{2.45} = 0.10
\]

\[
w_{\text{sand}} = \frac{1.2}{2.45} = 0.49
\]

The mix designer applies these weights to the individual material gradations to determine the combined gradation, then compares the combined gradation to the theoretical optimum gradation. The degree of mismatch is quantified by the sum of squared errors between the combined gradation and the theoretical target. For the mixture proportioned with 1-part cement : 0.25-parts silica fume : 1.2-parts sand, the error of the mixture is 168. Based on the comparison shown in Figure A.3.2.4-3, the mixture contains too many particles between 10 and 100 microns (0.0004 to 0.004 inches). The mix designer adjusts the proportions until the “error” is minimized and the silica fume content does not exceed 25 percent by weight of cement. For the three materials selected, the minimum error of 116 is obtained at proportions of 1-part cement : 0.25-parts silica fume : 1.37 parts sand.
Iterative trial batching is then performed in the laboratory for mixtures based on the "optimum" proportions. Various water contents and admixture dosages are combined with the solid materials until the desired properties (e.g., flow spread, compressive and flexural strength) are achieved. The mix designer selects the mixture that achieves the most favorable combination of workability, strength, and material cost for further development at the production scale.

A.3.2.3. Determining Water-Binder Ratio

A typical water-cement ratio (w/c) for UHPC is about 0.25 or less, which corresponds to a typical water-binder ratio (w/b) of less than 0.20. For the purposes of these Guidelines, "water" consists of all batched ice and water, any free moisture added from (or absorbed by) the aggregates, and the liquid water portion of chemical admixtures. The "binder" consists of the combined mass of the cement, silica fume, and any supplemental materials used.

An example w/c and w/b determination is provided below for the hypothetical UHPC mixture given in Table A.3.2.3-1. Note that the weight of silica flour is included in the total weight of the binder, even though this material is traditionally considered an inert filler for conventional concrete.
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Table A.3.2.3-1. Example Mix Proportions

<table>
<thead>
<tr>
<th>Material</th>
<th>Amount (lb/yd$^3$)</th>
<th>Free Moisture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>1335</td>
<td>--</td>
</tr>
<tr>
<td>Silica fume</td>
<td>334</td>
<td>--</td>
</tr>
<tr>
<td>Fly ash</td>
<td>334</td>
<td>--</td>
</tr>
<tr>
<td>Sand</td>
<td>1643</td>
<td>+2.5%</td>
</tr>
<tr>
<td>Water</td>
<td>169</td>
<td>--</td>
</tr>
<tr>
<td>Ice</td>
<td>73</td>
<td>--</td>
</tr>
<tr>
<td>High-range water reducer (30% solids)</td>
<td>53.4</td>
<td>--</td>
</tr>
<tr>
<td>Steel fiber</td>
<td>263</td>
<td>--</td>
</tr>
</tbody>
</table>

1. Calculate the total weight of the cement and binder:
   
   Total weight of cement $= 1,335 \text{ lb}$
   
   Total weight of binder $= \text{Weight of cement} + \text{Weight of silica fume} + \text{Weight of fly ash}$
   
   $= 1,335 \text{ lb} + 334 \text{ lb} + 334 \text{ lb} = 2,002 \text{ lb}$

2. Calculate the total weight of water:
   
   Free moisture from aggregates $= 1,643 \text{ lb} \times (+2.5\%) = 41.1 \text{ lb}$
   
   Water from admixtures $= \text{Water from high-range water reducer}$
   
   $= 53.4 \text{ lb} \times (100\% - 30\%)$
   
   $= 37.4 \text{ lb}$
   
   Total weight of water $= \text{Weight of water} + \text{Weight of ice} + \text{Free moisture from aggregates}$
   
   $+ \text{Water from admixtures}$
   
   $= 169 \text{ lb} + 73 \text{ lb} + 41.1 \text{ lb} + 37.4 \text{ lb} = 320.5 \text{ lb}$

3. Calculate water-cement ratio (w/c):
   
   Water-cement ratio $= \frac{\text{Total weight of water}}{\text{Total weight of cement}}$
   
   $= \frac{320.5 \text{ lb}}{1,335 \text{ lb}} = 0.24$

4. Calculate water-binder ratio (w/b):
   
   Water-binder ratio $= \frac{\text{Total weight of water}}{\text{Total weight of binder}}$
   
   $= \frac{320.5 \text{ lb}}{2,002 \text{ lb}} = 0.16$
A.4. UHPC Qualification, Acceptance, and Informational Testing

Before a UHPC mixture is used for the production of precast UHPC elements, qualification testing should be performed to characterize the performance of the material, as batched by the plant, and to demonstrate that the material can meet all project specifications and design requirements. During this process, the UHPC mixture is batched using the same batching equipment and mixing procedures (see Chapter 4) anticipated for full-scale production, and tests are performed on samples cast at the point of discharge from the mixer. Qualification testing is performed to verify that the minimum project requirements can be met by a particular concrete mixture. Qualification testing is different from quality control or acceptance testing, in that qualification testing is performed prior to production to demonstrate that the materials, proportions, and batching methods have the potential to create UHPC that complies with specifications. Results from qualification testing may be submitted to the Project Engineer, DOT, or other specifying body, for example, to demonstrate that a proposed mix design under consideration can achieve the minimum required performance.

Acceptance testing is testing performed during the production of a UHPC element to verify that the UHPC used within the element - considering the specific lot of materials, batch proportions, batching, and production and placement methods - complies with the minimum project specifications. As such, test specimens for acceptance testing are generally produced at the point of placement. Acceptance testing is an integral part of a plant's quality control program and is discussed in greater detail in Chapter A.6.

Additional informational testing of the UHPC may be performed at the discretion of the producer or designer. Informational testing includes evaluation of the UHPC performance in the plastic and/or hardened states to obtain any additional information related to production, mechanical performance, and/or long-term durability that is not required by the project specifications. Informational testing is different from qualification testing in that there are no specific design requirements to which the material must conform - tests are performed solely for informational purposes. Informational testing may be used to establish metrics for routine quality assurance (e.g., a “typical” flow or density that may be expected from a mixture), to provide more accurate values for design as further addressed in the Design Guidelines, or to verify adequate durability in aggressive environments (e.g., chloride penetration resistance testing).

A list of UHPC properties and recommended associated test methods are summarized in Table A.4-1. The typical purpose of the test, whether for mixture qualification, acceptance, or optional informational testing, is also identified. These properties and tests are further described in the following sections. Multiple tests are often required to properly characterize the expected performance and robustness of the mixture to qualify it for structural use.

Table A.2-2 summarizes the recommended minimum acceptance criteria for UHPC based on local materials for use in precast, prestressed elements. Additional qualification tests or stricter acceptance criteria may be required by local standards or individual project specifications. Additional product verification testing, which considers the impact of transport and placement methods on the mechanical performance of the UHPC element, is discussed in Section A.6.5.
## Table A.4-1. Test Methods for Precast, Prestressed UHPC

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Purpose of Test</th>
<th>Section*</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Plastic Properties</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td>ASTM C1064</td>
<td>X</td>
<td>A.4.1.1</td>
</tr>
<tr>
<td>Flow Spread</td>
<td>ASTM C1856</td>
<td>X</td>
<td>A.4.1.1</td>
</tr>
<tr>
<td>Working Time</td>
<td>ASTM C1856, measured over time as described below</td>
<td>--</td>
<td>A.4.1.1</td>
</tr>
<tr>
<td>Density (Unit Weight)</td>
<td>ASTM C138, with modifications described below</td>
<td>X</td>
<td>A.4.1.1</td>
</tr>
<tr>
<td>Time of Set</td>
<td>ASTM C1856</td>
<td>--</td>
<td>A.4.1.1</td>
</tr>
<tr>
<td>Fiber Segregation</td>
<td>See discussion below</td>
<td>X</td>
<td>A.4.1.1</td>
</tr>
<tr>
<td><strong>Hardened Properties</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>ASTM C1856</td>
<td>X</td>
<td>A.4.1.2</td>
</tr>
<tr>
<td>Flexural Performance</td>
<td>ASTM C1856</td>
<td>X</td>
<td>A.4.1.2</td>
</tr>
<tr>
<td>Static Modulus of Elasticity</td>
<td>ASTM C1856</td>
<td>--</td>
<td>A.4.1.2</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>ASTM C1856</td>
<td>If specified</td>
<td>A.4.1.2</td>
</tr>
<tr>
<td>Creep in Compression</td>
<td>ASTM C1856</td>
<td>If specified</td>
<td>A.4.1.2</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>ASTM C1856, with initial reading at final set or 24 hours, whichever occurs later</td>
<td>If specified</td>
<td>A.4.1.2</td>
</tr>
<tr>
<td>Chloride Content</td>
<td>ASTM C1218</td>
<td>If specified</td>
<td>A.4.1.3</td>
</tr>
<tr>
<td>Chloride Penetration Resistance</td>
<td>ASTM C1856 or ASTM C1556, performed on specimens without fibers</td>
<td>X (for structures exposed to chlorides)</td>
<td>A.4.1.3</td>
</tr>
<tr>
<td>Sulfate Resistance</td>
<td>See discussion below</td>
<td>If specified</td>
<td>A.4.1.3</td>
</tr>
<tr>
<td>Freeze-Thaw Resistance</td>
<td>See discussion below</td>
<td>If specified</td>
<td>A.4.1.3</td>
</tr>
<tr>
<td>Alkali Silica Reaction (ASR)</td>
<td>See discussion below</td>
<td>If specified</td>
<td>A.4.1.3</td>
</tr>
</tbody>
</table>

*Section of these Guidelines where tested properties are discussed.

### A.4.1. UHPC Properties and Test Methods

The following sections describe properties of UHPC mixtures important for the design and production of structural UHPC elements and identify test methods that may be used to characterize those properties. Mixture qualification is based on the minimum properties specified in Table A.2-2; while other properties described may be characterized for informational purposes, they are not required for design of UHPC elements according to the design guidelines.

#### A.4.1.1. Plastic Properties

The plastic properties of UHPC are influenced by the quality of the materials, the mixing procedures used, and the environmental conditions (e.g., ambient temperature, relative humidity) at the time of batching. Fresh UHPC should be sampled for each batch to verify that the particular combination of materials, mixing
procedures, and environmental conditions are yielding the desired performance. It is critical to characterize
the performance of the UHPC in its plastic state so that unacceptable material can be identified and remedied
or rejected before it is placed in formwork.

- **Temperature:** Monitoring temperature may be useful throughout the mixing process, but at minimum,
temperature should be measured and recorded at the time of discharge from the mixer and at the time of
placement, if placement occurs more than 15 minutes after discharge. The temperature of UHPC should be
measured using an immersible thermometer in accordance with ASTM C1064. Material temperatures
should be maintained between 50 and 80 °F (10 and 27 °C) during production of all structural precast
UHPC elements, and during production of specimens for qualification testing. At temperatures near or
below 50 °F (10 °C), setting times may be significantly extended; at temperatures near or above 95 °F (35
°C), the workability and working time of UHPC may be significantly reduced and the surfaces may be more
difficult to finish. If qualification samples are cast from multiple batches of UHPC, it is recommended that
the temperatures of each batch differ by no more than ± 5 °F (± 3 °C).

- **Flow spread:** Initial acceptance or rejection of a batch of UHPC is based on its flow spread. Flow spread is
a measure of the workability of the UHPC, similar to the slump of a conventional concrete mixture or the
flow spread of a self-consolidating concrete (SCC) mixture. Unlike the SCC flow spread test, the UHPC flow
spread test is performed using a cone meeting the requirements of ASTM C230. This cone is smaller than
a standard concrete slump cone, with a height of 2 inches (50 mm), a top opening diameter of 2.75 inches
(70 mm), and a bottom opening diameter of 4 inches (100 mm). UHPC is added to the cone in a single layer
as described in ASTM C1856, without rodding. The cone is lifted, and the material is spread out onto a
clean, smooth, and level surface. After 2 minutes, the largest and smallest diameters of the spread are
measured to the nearest 1/16 inch (1 mm). The average of the two measurements is recorded as the flow
spread of the material.

At a minimum, flow spread should be measured for each batch within 15 minutes of placement. It may be
desirable, particularly during trial batching or for the first batch in a day’s production, to also measure flow
spread prior to the addition of the fibers, so that the acceptability of the batch can be confirmed before
incorporating the typically expensive fibers.

For fluid (self-consolidating) UHPC, the flow spread of the UHPC should be between 8 and 10 inches (200
to 250 mm) at the time of placement. Placement into the forms and fabrication of all laboratory test
specimens should be initiated within 15 minutes of the flow measurement, otherwise an additional
measurement should be performed. UHPC with flow spread of less than 8 inches (200 mm) is more difficult
to place and more likely to entrap air voids during placement into the forms, while UHPC with flow spread
of more than 10 inches (250 mm) is more likely to segregate. UHPC with flow spreads exceeding 10 inches
may be used, provided it can be demonstrated that the constituent materials do not segregate (see “Fiber
segregation” below). If flow spread is less than 8 inches, it may be permissible to add additional HRWR or
water to the mixer, provided that the maximum allowable water content of the approved mix design is not
exceeded.

It has been observed that when fibers are added too rapidly to a UHPC mixture having high flow spread of
about 8 inches (200 mm) or more, there is a tendency for the fibers to clump rather than disperse uniformly
throughout the mixture. Therefore, it may be desirable to add fibers to UHPC having a lower flow spread,
between about 6 and 8 inches (150 to 200 mm), to facilitate the breaking up of fiber clumps during mixing,
then adding a reserve of water or HRWR to the mixture to increase the flow spread to target placement
levels.

- **Working time:** The working time of UHPC is the time that the material remains sufficiently flowable for
product fabrication. For most precast applications, “sufficiently flowable” means that the flow spread
remains above 7 inches (175 mm). Working time is typically evaluated for information purposes only and
is not a qualification or acceptance criteria; nevertheless, it is expected that a UHPC mixture will remain
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workable for the entire placement of an element, even if the element is placed using multiple batches of material. The working time will be influenced by mixture constituents, particularly the HRWR and other chemical admixtures, as well as the environmental conditions, such as temperature. A working time of 1 to 2 hours can be reasonably achieved for most UHPC mixtures; longer working times may be achieved through the use of certain chemical admixtures, such as workability retention admixtures and set retarders, or through cooler UHPC temperatures, as may be achieved by substituting a portion of the mixing water with ice.

Working time can be measured for informational purposes during mixture qualification or other trial-batching efforts. There is no standardized testing method, but working time may be determined by measuring the flow spread at the time of discharge from the mixer, then periodically (e.g., every 15 minutes) thereafter until two successive measurements of less than 7 inches (175 mm) are recorded. The working time is the time at which the flow spread equals 7 inches (175 mm) as determined by linear interpolation. When evaluating the working time of a particular mixture, it is recommended that the UHPC be handled in a manner that is consistent with the anticipated handling and transport methods to be used for production. For example, if it is anticipated that the UHPC will be placed into the forms from a mixing truck, the material used for working time evaluation should be sampled from a mixing truck or other mixer that can adequately simulate the rotational mixing action of a mixing truck.

• **Density (unit weight):** Density measurements are recommended for routine quality control of UHPC to quickly identify changes in mixture quality. It is recommended that density be measured at the start of each day’s production and whenever quality control specimens are cast. It can also be performed any time a change in materials or quality is suspected. Density should be measured at the point of discharge from the mixer and in accordance with ASTM C138, with the exception that the unit weight measure should be filled in a single layer without rodding or vibration. The measure should be consolidated by tapping the sides of the mold 30 times with a mallet, struck-off smooth, and cleaned prior to weighing. The measured density should be compared to previous measurements of density for the same mixture and to the theoretical unit weight of the material to identify potential changes in mixture quality. The density should not deviate from the established target value by more than 2.0 lb/ft^3 (32.0 kg/m^3) (Precast/Prestressed Concrete Institute, 1999). Observed changes in density may indicate a change in entrapped air content, a change in aggregate moisture content, an error in batching weights, or segregation of fibers or other constituent materials.

• **Time of Set:** Understanding the setting behavior of a UHPC mixture can be beneficial for comparing materials or mix designs considered for use in structural UHPC elements, or for timing shrinkage measurements (see Section A.4.1.2). According to ASTM C1856, the setting time of UHPC is evaluated using a Vicat apparatus in accordance with ASTM C191, with the exception that the UHPC is batched at its specified water content and the mold is filled without consolidation. If a potential for metallic fibers to impede the motion of the Vicat test needle is identified, setting time measurements may be performed on specimens without metallic fibers. Because of the typically large dosages of HRWR used in these mixtures, final setting times are often longer for UHPC than for conventional concrete and may exceed 24 hours for certain admixture combinations (ACI Committee 239, 2018). Setting time is typically not specified as an acceptance criterion for UHPC, but is recommended to be evaluated for informational purposes during trial batching. Because setting time is sensitive to temperature, it may be desirable to perform testing under anticipated temperatures for production, rather than under the controlled environmental conditions as specified in ASTM C191.

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2 Practically for field production, it is usually only useful to know that final setting of the UHPC has occurred. Final setting is defined by ASTM C191 as occurring when the Vicat needle “does not mark the specimen surface with a complete circular impression.” Application of heat curing, for example, should not occur until after the UHPC has achieved final set.
Fiber segregation and dispersion: The stability of the UHPC and the potential for segregation and/or limited dispersion should be considered for highly fluid mixtures, especially for those with flow spreads in excess of 10 inches (250 mm). These properties will be impacted by the process in which the fibers are introduced into the mix; rapid addition may result in fiber clumps that the mixing action cannot break up (see discussion in Section A.5.2.2). While there are currently no standardized methods for evaluating fiber segregation and dispersion, three approaches may be considered. The first approach is to observe the distribution of fibers as the material flows during a flow spread test. UHPC mixtures with insufficient fiber dispersion or with a high tendency to segregate may retain a portion of fibers within the center of the spread, or exhibit fiber clumping within the flow spread, as shown in Figure A.4.1.1-1.

The second approach is a modified static segregation test, based on ASTM C1610 for self-consolidating concrete. In this test, the UHPC is poured into a 26-inch (660 mm) tall column. After 15 minutes of settlement time, the material in the top 6.5 inches (165 mm) and bottom 6.5 inches (165 mm) of the column are removed. The fibers are separated from the rest of the UHPC paste by sieving, then cleaned and weighed. For steel fibers, magnets may be used to help separate the fibers. The difference in weight between fibers in the top 6.5 inches and the bottom 6.5 inches are compared. A difference of more than 10 percent may indicate fiber segregation.

The third approach to characterizing fiber segregation is to pour the UHPC into a mock-up of similar height to the element to be cast. The UHPC is poured into the mock-up element and consolidated as intended. After the UHPC has hardened, the element is cut vertically and microscopic evaluation is used to quantify the amount of fibers in the top and bottom 1/4 of the mock-up. Like the static segregation test, a difference of more than 10 percent fiber volume between the upper 1/4 of the mock-up and the lower 1/4 of the mock-up may indicate unacceptable fiber segregation.
Figure A.4.1.1-1. Flow Spread Tests Performed on UHPC Mixtures with Well-Distributed Fibers, and (b and c) with Varying Tendencies for Fiber Clumping and Segregation

A.4.1.2. Hardened Properties

Much of the structural design of UHPC relies on the hardened properties of the material. The design principles and equations presented in the design guidelines are based on the concrete's minimum specified compressive and flexural properties, which must be satisfied (through both qualification testing prior to production and acceptance testing during production) by the specific mixture used for a UHPC element. For UHPC elements designed in accordance with the design guidelines, qualification of a UHPC mixture is based solely on the compressive strength and flexural performance of a particular mixture. Measurement of other mechanical properties such as modulus of elasticity, Poisson's ratio, creep coefficient and shrinkage is not required for qualification or acceptance when an element is designed according to the design guidelines; however, these parameters may also be characterized for the mixture for informational purposes and used in the design.

The hardened properties, particularly the strength at prestress release and initial shrinkage, and the long-term creep and ultimate strength are strongly influenced by curing temperatures, and therefore, all materials qualification and acceptance tests should represent the anticipated or actual curing and thermal treatment methods used by the precasting facility. Due to the inherent variability in materials, mixture proportions, and processes used to produce UHPC, qualification testing should be performed on a statistically significant number of samples, preferably from multiple batches of material (Canadian Standards Association, 2019; Association Francaise de Normalisation (AFNOR), 2016). A large number of samples (30+) is recommended for establishing appropriate statistics and reliability metrics for a particular mix design and production process. Performance reliability and statistical analysis of material performance characteristics are discussed in greater detail in Section A.4.3.
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- **Compressive strength**: The compressive strength of UHPC is highly dependent on the constituent materials, mixture proportions, and curing conditions. UHPC mixtures based on local materials have been developed with 28-day compressive strengths ranging from 18,000 psi (125 MPa) to more than 30,000 psi (205 MPa) (Wille, Naaman, and El-Tawil, 2011). Compressive strength is generally higher for UHPC that has been cured with heat or thermally treated (see Section A.5.5.1), due in part to the acceleration of the hydration reaction, and in part to the increased efficiency of the pozzolanic reactions of the silica fume and other SCMs at elevated temperatures (Wang, et al., 2015). It is important that the curing regime, including any post-curing thermal treatments, be documented when performing qualification testing for a given UHPC mixture, and that the same curing regime be used for the production of corresponding structural elements as for the qualified mixture. If a mixture can be cured via multiple regimes or thermal treatment procedures, compressive strength should be independently qualified for each permissible combination of curing regime and thermal treatment.

Compressive strength should be evaluated according to ASTM C39, with modifications defined in ASTM C1856. Testing is performed on 3-inch by 6-inch (75 by 150 mm) cylindrical specimens, loaded at a rate of 145 psi per second (1 MPa per second) until failure. Prior to testing, the ends of the cylinders should be ground flat and plane to avoid reductions in observed compressive strength caused by surface irregularities (Schmidt and Frohlich, 2010). Testing with other surface preparation methods such as capping is not recommended for compressive strengths above 12 ksi (83 MPa) due to the increased potential for measuring unrepresentative strengths: testing at these levels with bonded caps may produce low strength results, while testing with unbonded caps may produce high strength results. However; these alternate methods may still be useful for early-age strength testing, before the concrete as reached a compressive strength of 12 ksi (83 MPa).

For qualification of a new UHPC mixture, testing should be performed for a minimum of 30 specimens at an age of 28 or 56 days, whichever is specified by the owner of the structure or its representative. Specimens should be sampled from at least three separate batches, with 3 to 10 specimens sampled from each batch. Specimens should be cured in the same manner as anticipated for the structural elements. The average and standard deviation of measured test results should be determined for each mixture and curing condition. For qualification, the average measured compressive strength minus 1.4 times the measured standard deviation should be greater than the required (f'_c) compressive strength. The value calculated in this way represents the characteristic strength of the mixture for the production process used and corresponds to a statistical 95% probability that this value will be met or exceeded during production (Canadian Standards Association, 2019). The characteristic strength used for qualification may be updated based on field performance once 30 consecutive compressive strength tests have been performed for a given mixture.

For additional mixture characterization, compressive strength testing at 1, 2, 3, 7 and 14 days (and 28 days if 56 days is basis for specified strength) of age should be considered to establish the rate of strength gain that may be anticipated during production so that the age for pretension release can be estimated for planning purposes. Producers may also consider performing maturity testing (ASTM C1074) to estimate the in-place compressive strength of precast UHPC elements during production, based on temperatures measured in the elements.

For acceptance testing, specimens may be match-cured (AASHTO R72) with the product, if desired. Because UHPC mixtures develop strength significantly more rapidly when exposed to elevated temperatures from heat curing or from the development of high internal temperatures due to heat of hydration, match-curing tends to provide higher compressive strengths at a given age than standard ambient-temperature curing.

- **Flexural performance**: The tensile performance of UHPC is characterized by a high cracking strength and post-cracking ductility. Design for tensile strength of UHPC is an important feature, and it is critical that qualification testing demonstrate that a particular UHPC mixture possesses the required tensile
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Performance at cracking and the residual post-cracking strength assumed by design. Tensile performance may be characterized by direct tensile testing (Graybeal and Baby, 2013), but this method is impractical for routine usage. It is more convenient to measure tensile performance through flexural testing of the UHPC. The design guidelines describe the relationships between the measured flexural performance and the flexural and shear response that may be assumed in the design of UHPC structural members.

Flexural performance should be measured under four-point bending according to ASTM C1609, using prismatic specimens sized according to ASTM C1856 based on the length of the fibers. It is important that the test fixture have freely-rotating supports to permit rotation of the specimen after cracking, and that the loading frame be equipped with a servo control mechanism (per ASTM C1609) to control the rate of loading after the first crack has initiated. Restraint of specimen rotation by non-rotating supports may erroneously increase the measured flexural strength by up to 40 percent (Wille and Parra-Montesinos, 2012), while failure to control the load rate after first cracking may prevent accurate measurement of the peak (ultimate) flexural strength and the residual flexural strength of the material. It is also important that specimens be fabricated in a consistent manner, filling the molds in a continuous manner from a single point of placement at one end of the mold, in accordance with ASTM C1856. While this method of sample fabrication may result in preferential alignment of fibers along the tensile failure surface of the specimen and an increased strength versus samples with randomly oriented fibers, the intent is to produce consistent samples for testing. (See Section A.6.5 for additional discussion regarding evaluation of fiber alignment in mock-up elements.)

For mixture qualification, flexural testing should be performed and interpreted in a similar manner to that for compressive strength qualification testing, with similar requirements for number of tests and sample curing. A full flexural strength versus deflection curve should be generated for each specimen to properly characterize the first-peak (first-crack) strength, peak (ultimate) strength, and residual strengths at a deflection of L/300 and L/150, where L is the span length of the flexure beam. Flexural toughness (the energy absorbed by the sample during testing, which can be quantified as the area under the load-deflection curve) may also be characterized for informational purposes. The average and standard deviation of measured test results should be reported for each mixture and curing condition and used to establish a characteristic strength.

For further mixture characterization, additional flexural testing at the anticipated age of prestressing release may be considered to verify that the mixture achieves the first-peak (first-crack) flexural strength at release required by the prestressing design.

- **Modulus of elasticity**: The static modulus of elasticity of UHPC is typically considerably higher than for conventional concrete, consistent with its higher compressive strength. Typical values reported in the literature range between 6,000 and 8,700 ksi (40 and 60 GPa) at 28 days, depending on the strength of the UHPC and the curing regime used (Russell and Graybeal, 2013; ACI Committee 239, 2018; Swiss Society of Engineers and Architects (SIA), 2016). The static modulus of elasticity may be evaluated for informational purposes according to ASTM C469, as modified by ASTM C1856; however, testing is not required for qualification of a UHPC mixture but may be useful for design based on the design guidelines. If performed, testing should be done on 3-inch by 6-inch (75 mm by 150 mm) cylinders cast in a single lift as for compressive strength testing, and cured in a like manner as the compressive strength cylinders.

- **Poisson’s ratio**: Poisson’s ratio is typically measured concurrent with static modulus of elasticity, in accordance with ASTM C469, as modified by ASTM C1856. Like static modulus of elasticity, measurement of Poisson’s ratio is not necessary for design based on the design guidelines, but may be performed if requested. Poisson’s ratio for UHPC has been measured between 0.16 and 0.21, with typical values around 0.18 to 0.20 (Russell and Graybeal, 2013; Swiss Society of Engineers and Architects (SIA), 2016). In the absence of a measured Poisson’s ratio, a value of 0.20 may be assumed.
Creep: The creep behavior of UHPC is dependent upon the curing regime used and the age of the concrete at the time of loading. UHPC that has been heat-cured has been shown to have significantly reduced long-term creep compared to UHPC that has been cured under ambient conditions (Russell and Graybeal, 2013; Swiss Society of Engineers and Architects (SIA), 2016; Garas, Kahn, and Kurtis, 2009; Garas, 2009). The decrease in long-term creep is likely associated with the more advanced hydration of the cementitious materials, coupled with the consumption of water from the UHPC porosity during the high-temperature curing (Neville, 2011).

The creep coefficient is defined as the ratio of the long-term creep strain to the initial elastic strain. Compressive creep coefficients reported in the literature for UHPC loaded at 28 days range from 0.6 to 1.4 at 1 year for UHPC that has not been heat-cured (Fehling, et al., 2014) and 0.2 to 0.5 at 1 year for UHPC that has been heat-cured (Gowripalan and Gilbert, 2000; Association Francaise de Normalisation (AFNOR), 2016). For comparison, compressive creep coefficients of conventional concrete generally range between 1 and 2 at 1 year of age (Mehta and Monteiro, 2006). To date, most research has considered compressive creep, with limited research performed on tensile creep. The limited research available indicates that tensile creep of UHPC may be up to 10 times greater in magnitude than compressive creep (Garas, 2009).

Shrinkage: Shrinkage of UHPC has two primary components: autogenous shrinkage, which is caused by the consumption of water by the reaction of the cementitious materials, and drying shrinkage, which is caused by the gradual loss of moisture from the concrete to the environment. In most conventional concrete, autogenous shrinkage is negligible because the hydration reactions do not usually consume enough water to generate large shrinkage strains; however, in UHPC, the low water-cement ratios, high cementitious materials contents, high silica fume dosages, and small maximum aggregate size make autogenous shrinkage a significant contributor to the overall volume change of the UHPC. In fact, because there is generally very little moisture remaining in the pores to cause drying shrinkage, most of the volume change of a UHPC element is driven by autogenous shrinkage and not by drying shrinkage (Russell and Graybeal, 2013). Typical magnitudes of autogenous shrinkage for UHPC range from 200 to 1200 microstrain, while typical magnitudes of drying shrinkage range from 0 to 600 microstrain, with near-zero levels of drying shrinkage occurring after heat curing (Haber, et al. 2018; this study). The magnitudes of autogenous and drying shrinkage may be reduced through the use of shrinkage reducing admixtures (SRAs) or offset through the use of expansive agents based on periclase (MgO) or lime (CaO) in the mix design (Soliman, 2011). Appropriate dosages of these materials would need to be determined by trial batching.

Shrinkage is not typically specified for mixture qualification, but may be requested for informational purposes. Autogenous shrinkage may be measured in the laboratory according to ASTM C1698, but it is typically more useful to measure the total combined autogenous and drying shrinkage of a UHPC using a modified ASTM C157 (ASTM C1856) drying shrinkage test, performed on 3-inch by 3-inch by 11.25-inch (75 mm by 75 mm by 285 mm) prisms. To measure the total combined autogenous and drying shrinkage, the first length measurement should be made at the time of final setting, with subsequent length-change measurements made, at minimum, after 1, 2, 4, 7, 14, 28, 56, and 90 days of drying.
A.4.1.3. Durability

In general, UHPC possesses exceptional long-term durability characteristics due to its low porosity and low permeability to moisture. Because UHPC may be used in aggressive environments, a UHPC mixture may be required to demonstrate satisfactory performance by one or more durability-related tests during mixture qualification. The most commonly specified durability-related characteristics are discussed below.

- **Chloride content**: The total chloride content of UHPC may be requested to verify that chloride ions within the UHPC materials do not substantially increase the risk of corrosion of the reinforcing steel or strand. If required, water-soluble chloride content may be evaluated according to ASTM C1218 for each constituent material individually or for a hardened UHPC sample between 28 and 42 days of age. For prestressed concrete, the total water-soluble chloride ion contributed by the constituent materials, or present in the hardened UHPC sample, should be less than 0.06 percent by weight of cement, as recommended by ACI 318-14 (ACI Committee 318, 2014).

- **Chloride penetration resistance**: A particular level of chloride penetration resistance may be specified for structural UHPC elements subject to marine or deicer exposures or in contact with chloride-containing soils or groundwater. Chloride penetration resistance may be evaluated by a variety of methods, but is most commonly evaluated by ASTM C1202 / AASHTO T277 (rapid chloride permeability [RCP] test) or ASTM C1556 (bulk diffusion test). The RCP test provides a rapid indicator of concrete’s ability to resist chloride ion penetration, while the bulk diffusion test provides a more direct measurement of the concrete’s apparent chloride diffusion coefficient, which may be useful for service life prediction. As with many UHPC properties, the results of these tests are dependent on the curing regime and thermal treatment used and on the age of the concrete specimens at the time of testing. Values reported for the RCP test conducted on standard, laboratory-cured UHPC have been more than five times larger than values reported for UHPC specimens receiving a secondary post-cure thermal treatment (Russell and Graybeal, 2013). Chloride diffusion coefficients measured by ASTM C1556 or AASHTO T259 are generally two orders of magnitude smaller than those reported for conventional concrete (Thomas, et al., 2012; Russell and Graybeal, 2013). For UHPC elements designed according to the design guidelines and subject to a chloride exposure, qualification of the mix design is based on the ASTM C1202 / AASHTO T277 RCP test. It is recommended that UHPC have a total charge passed less than 500 Coulombs at 28 days when tested according to this method.

When characterizing the chloride penetration resistance of UHPC, some modifications to testing procedures may be required. In particular, electrical tests, such as the RCP test, are affected by the presence of metallic fibers and may produce erroneous measurements or damage testing equipment if performed on specimens with a significant quantity of metallic fibers. Accordingly, RCP test samples should be prepared without the inclusion of metallic fibers.

- **Sulfate attack resistance**: The sulfate attack resistance of UHPC is typically not of concern due to the low porosity, low moisture permeability, and high SCM content of the UHPC. In limited laboratory studies reported to date, little to no deterioration of UHPC was noted after exposure to sodium sulfate solution (Russell and Graybeal, 2013). A secondary form of sulfate attack, called thaumasite attack, may occur in systems containing calcium carbonate (typically coming from ground limestone) with extended periods of exposure to cool ambient temperatures around 50 °F (10 °C). While thaumasite has been noted to form in conventional concrete systems, it is unlikely to occur in concrete, including UHPC, that has adequate resistance to sulfate attack (Barcelo, et al., 2014).

- **Freeze-thaw resistance**: The freeze-thaw resistance of UHPC is generally very good compared to conventional concrete. When tested according to ASTM C666, Method A, as modified by ASTM C1856, most UHPC mixtures exhibit little to no deterioration after 300 cycles of freezing and thawing, despite generally having no air entrainment (Thomas, et al., 2012; Russell and Graybeal, 2013). Many UHPC mixtures actually show an increase in durability factor due to further hydration of the cementitious materials over the duration of the exposure. The improved freeze-thaw resistance of UHPC may be
attributed to the low porosity of the UHPC, which makes it difficult for the concrete to become critically saturated, and the high strength of the cementitious matrix, which makes it more resistant to cracking under the stresses generated by the freezing water (Neville, 2011).

- **Delayed ettringite formation (DEF):** DEF is an expansive reaction that can take place in certain types of concrete that have been exposed to elevated temperatures (above 158 °F [70 °C]) during hydration and/or curing. Because UHPC has a high cementitious materials content, it produces a large amount of heat during hydration, which may increase the internal temperatures of the UHPC above 158 °F (70 °C). In addition, common heat treatments for UHPC routinely involve exposing a UHPC test specimen or element to temperatures of up to 194 °F (90 °C) for 48 hours, which exceeds maximum temperature limits set in PCI MNL-116 and other documents for avoiding DEF in conventional concretes. The expansive reaction that causes DEF-related damage to concrete requires exposure to moisture; therefore, due to the low porosity of UHPC and its low permeability to water, DEF is not generally a concern (Heinz and Ludwig, 2004; Pfeifer, et al., 2009). In addition, the use of silica fume and other supplementary cementitious materials (SCMs) has been shown to reduce the risk of DEF (Precast/Prestressed Concrete Institute, 1999; ACI Committee 201, 2016). Therefore, provided that the UHPC contains silica fume and meets all specified permeability requirements, current understanding based on the available research indicates that it is unlikely to be susceptible to DEF-related distress.

- **Alkali-silica reaction (ASR):** ASR is an expansive reaction that takes place in concretes containing high alkali contents provided by the cementitious materials, a source of reactive silica provided by the aggregates, and access to moisture. Although UHPC mixtures tend to have larger total cementitious contents, and therefore potentially higher alkali loadings compared to other conventional concrete mixtures, ASR is not typically a concern for UHPC due to its low permeability to moisture and its typically high SCM contents. If avoidance of reactive aggregates is not possible as a means to eliminate the risk of ASR, the UHPC is still unlikely to experience ASR-related distress, provided that the UHPC meets all specified permeability requirements and contains sufficient quantity of silica fume.

### A.4.2. Robustness Evaluation

The performance of a UHPC mixture in both the fresh and hardened states may be highly sensitive to variations in mixture proportions, temperature, and constituent material properties, such as moisture content and particle size. A robust concrete mixture is one that can be produced to achieve consistent properties, despite minor variations in these characteristics (Daczko, 2012). A mixture with poor robustness will require additional quality control measures, such as continuous monitoring of aggregate moisture content, to ensure that variations in these parameters are appropriately accounted for during batching.

UHPC producers may wish to evaluate the robustness of a new UHPC mixture to better understand the relative sensitivity (or insensitivity) of the material’s performance to these various factors. The purpose of robustness testing is to identify the limits at which the mixture will no longer exhibit the intended properties in the fresh and hardened states. Examples of robustness evaluations may include:

- Evaluating the effect of HRWR dosage on flow spread, fiber segregation resistance, and workability retention. Tests may be performed on batches of UHPC containing multiple high and multiple low HRWR dosages until unacceptable performance is achieved or the point at which unacceptable performance is achieved can be predicted. This may facilitate more effective batch adjustments if undesirable performance is achieved during production.

- Evaluating the effect of temperature variation on admixture efficiency and workability retention. Tests may be performed on batches of UHPC produced with heated or chilled constituent materials, or in environments simulating a range of ambient temperatures that may be experienced during production. Temperature robustness evaluations may provide a basis for batch adjustments or other strategies required for production in hot or cold ambient temperatures, or better define the acceptable range of placement temperatures.
• Evaluating the effect of aggregate moisture content on batching times, flow spread, and mixture consistency. Tests may be performed on batches of UHPC with aggregates at different moisture states (e.g. dry, wet, very wet) to better understand the relative sensitivity of the material to variations in aggregate moisture. Note, aggregate moisture content should be properly accounted for in the total water content for this evaluation.

• Evaluating the effect of variable mixture proportions on flow spread and strength. Tests may be performed on batches of UHPC with higher or lower material quantities than specified in the target mix design. Typically, one constituent is varied at a time. For example, varying water content while keeping all other batch quantities fixed. For effective evaluation, variables should be examined at the upper and lower specified batching tolerances, and at values outside of the permissible range. When evaluating only one parameter at a time, interactions between variations (e.g., high water content and low cement content) are not examined; however, the benefit of this approach is that the materials having the greatest impact on the performance characteristics can be identified, and more careful control of those parameters can be incorporated into the plant’s quality control program.

This list is not exhaustive, but provides some suggestions for producers who wish to evaluate the robustness of a new UHPC mixture or troubleshoot problems with an existing mixture. For mixtures exhibiting poor robustness, improvements may be made by adjusting the mixture proportions, changing a material source to a more consistent product, or incorporating alternative or additional chemical admixtures into the mix.

A.4.3. Performance Reliability

Like all concrete materials, UHPC is a mixture of a variety of constituents whose properties and proportions may vary from batch-to-batch and even within a single batch. As a result, it can be expected that compressive strength, flexural strength, and other properties used as a basis for design will vary both between successive batches of UHPC and within individual batches of UHPC.

The reliability of a mixture in achieving a particular level of performance is typically characterized by statistical analysis of a series of at least 30 tests. Tests may be performed consecutively or may be pooled from two or more series of tests as defined by the governing specifications. The performance is characterized in terms of the average, standard deviation, and coefficient of variation of the test results, and statistical analysis is used to evaluate the “reliability” of the mixture. ACI 214R, Guide to Evaluation of Strength Test Results of Concrete (2011), provides a good overview of statistical analysis principles commonly used to characterize the performance of concrete materials.

Two approaches are conventionally adopted to evaluate performance reliability. In the United States and Canada, performance reliability is factored into materials acceptance criteria. For example, ACI 318 gives materials acceptance criteria based on the specified strength for design (e.g., $f'_c$), the variability of the test results (e.g., standard deviation of test results), and the proportion of tests allowed to be below the specified strength (e.g. 1 in 100). The minimum required average value for materials acceptance (e.g., minimum required average compressive strength $f_{cr}$) is then determined as the specified design strength plus a factor that accounts for the expected sample variation. In other words, in the United States and Canada, the producer must demonstrate that the average strength of their material exceeds the design strength by a certain specified amount.

In Europe and elsewhere, an alternative but parallel approach is adopted. Rather than requiring that the average material strength exceed the design strength by a specified amount, in Europe and other countries, the “characteristic” strength of the material must exceed the design strength. The “characteristic” strength is essentially the strength level that can be reliability achieved by 95 percent of future tests on the material. It is calculated by reducing the average measured strength by a factor that accounts for the expected sample variation, similar to the factor used to determine the required average strength in the United States and Canada.

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A.5. Production of UHPC

Structural elements constructed with UHPC mixtures based on locally-available materials can be produced using similar equipment as conventional concrete; however, there are a number of significant differences in the batching process and in the placement, finishing, and curing procedures that are used. This chapter discusses the particular aspects of UHPC production that should be considered by designers, specifiers, and producers of structural UHPC elements.

A.5.1. Constituent Materials Handling

As discussed in Chapter 2, the primary constituents of UHPC are cement, silica fume, supplemental material(s), fine aggregate, chemical admixtures, and fibers. The dry constituents (typically the cement, silica fume, supplemental materials, aggregates, and possibly fibers, as well) may be pre-blended and stored as a unit, or stored in individual silos or storage areas and combined during batching. Constituent materials should be stored in a manner consistent with PCI MNL-116 (Precast/Prestressed Concrete Institute, 1999). Materials should be stored in a way that protects them from contamination or impurities. Materials stored in bags or sacks should be stored in a dry location with adequate drainage. If moisture exposure is anticipated from the ground, bagged materials should be stored in an elevated location, such as on pallets or racks, to reduce exposure to moisture from the ground.

Cementitious materials should be stored in a dry area to limit pre-hydration. Cementitious materials that develop hard lumps due to partial hydration should not be used. Handling and storage of pre-blended materials should be performed in a way that minimizes segregation of the constituent materials.

Aggregates should be stored in enclosed bins or silos, when possible, to provide greater uniformity and control over the aggregate moisture content. Aggregates may be used in a wet or oven-dry state, but more frequent testing of moisture content or more careful stockpile management may be required for aggregates stored in a wet condition (see Section A.5.2.3). For this reason, some producers have preferred to store aggregates for UHPC in an oven-dry condition; however, availability of dry aggregates may be limited or cost prohibitive in some locations, as aggregates are not normally dried for concrete production. In addition, aggregates stored in a dry condition may be more susceptible to segregation (Precast/Prestressed Concrete Institute, 1999).

Chemical admixtures should be stored in accordance with the manufacturer’s recommendations. At minimum, admixtures for UHPC should not be exposed to freezing temperatures. Admixtures should not be used past their maximum shelf-life without prior consultation with a representative of the admixture manufacturer.

Fibers should be stored in a dry, covered location to prevent oxidation (metallic fibers) or UV degradation (non-metallic fibers). Some surface oxidation (rust) of metallic may be permissible, provided that the fibers remain as individual strands (i.e., do not clump) and the severity of the oxidation is documented. Discoloration of non-metallic fibers may indicate UV degradation; therefore, discolored fibers should not be used.

To control mixing temperatures, particularly in warm climates or in summer months, it is recommended that constituent materials be stored in such a manner that they are between 50 and 95 °F (10 and 35 °C) at the start of mixing. Storage in a cool, shaded location is encouraged when possible. If cementitious materials are delivered to the plant in a warm condition, it is recommended that they be allowed to cool to less than 95 °F (35 °C) prior to batching; if such cooling is not viable, use of chilled water, ice, or other cooling measures during mixing should be considered.

A.5.2. Mixing

The mixing process for UHPC typically consists of three phases: in the first phase, constituent materials are dry-mixed to achieve homogenization in the mixer; in the second phase, water and chemical admixtures are added until a sufficiently fluid mixture is obtained; and in the third phase, fibers are added to create the composite. Alternative mixing procedures, with fibers added during the dry-mixing phase or interspersed with the water and chemical admixtures, have also been used, but may require additional considerations. In all cases, the mixing sequence used should aim to ensure uniform dispersion of the constituent materials, uniform distribution of the fibers within the mixture, and achievement of the fresh properties required for transporting
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and placing the UHPC. Trial batching is recommended. Mixing procedures that result in clumping of the fibers or cementitious materials should be avoided.

The following guidance is provided to assist with the development of a batching procedure, but it is ultimately up to the UHPC plant operator to determine the batching volumes, mixing times and sequence of material additions to the mixer appropriate for their equipment and UHPC mixture.

A.5.2.1. Mixer Requirements

UHPC can be produced in almost any type of concrete mixer, from the rotating drum of a transit mix truck to the higher energy horizontal shaft or planetary mixers. There can be significant differences in the efficiency and effectiveness between different types of mixers. Because of the low water-cement ratios, the high fluidity of a UHPC mixture is achieved primarily through the use of HRWR and the energy imparted to the materials by the mixer. A significant amount of mixing energy, which correlates to relatively longer mixing times, is required to overcome the internal shear friction between the particles to convert the UHPC from a collection of dry solids into a fluid mixture (ACI Committee 239, 2018).

While nearly any mixer can be used to produce UHPC, the total duration of mixing will be largely governed by the energy imparted by the mixer. High-energy mixers can be used to produce UHPC within a few minutes, while lower energy mixers and conventional concrete mixing trucks may require 30 minutes or more before the UHPC turns into a fluid mixture. Although mixing in a mixing truck is possible, it is not recommended due to the lower mixing efficiency and the extended mixing times that may be required. UHPC produced in a typical central batch plant mixer for precast production will likely turn to a fluid within about 5 to 10 minutes of total mixing time after the water and HRWR have been combined with the dry materials.

One important consideration of UHPC production that differs from conventional concrete production is batch size. The bulk volume of the constituent materials for UHPC is about twice the volume of the fresh UHPC (Swiss Society of Engineers and Architects (SIA), 2016). In addition, a large amount of torque is required to mix the dry materials and then transition the mixture from a dry solid material to a fluid mixture. These two factors may limit batch sizes for UHPC to less than the nominal capacity of the mixer (e.g. the mixer may only be able to handle one-half to two-thirds the nominal capacity). Trial batching may be the best process for determining the optimal batch size for a particular mixer.

A.5.2.2. Mixing Sequence

The condition of the mixer at the start of mixing should be consistent for each batch. This may require either “buttering” the mixer prior to production of the first batch of the day, or cleaning of the mixer between successive batches. “Buttering” the mixer is intended to produce a lining of paste representative of the condition of the mixer after each batch is dumped. Subsequent batches can be produced without cleaning the mixer, provided that the new material is charged into the mixer within the working time of the previous batch (i.e., while the “buttering” material is still fluid). Cleaned mixers should be in a saturated, surface-dry (SSD) condition without visible pools of water.

UHPC mixing is typically performed in three phases. The first phase consists of dry-mixing the sand and the powder materials until a homogeneous blend is obtained. If using dry, densified silica fume, it is recommended that the fine aggregates and silica fume are added first, followed by the cement and supplemental materials. Dry-mixing of densified silica fume with the fine aggregates for at least one to two minutes prior to addition of the remaining powder materials may improve dispersion of the agglomerated silica fume particles. Dry materials should be mixed for a sufficient period of time, typically one to two minutes, to ensure uniform blending and dispersion prior to the addition of any liquid components. If using a silica fume slurry, it is recommended that the slurry be added after all other dry materials have been added to the mixer.

During the “dry-mixing” phase, it may be beneficial to add a portion of the water and HRWR (about one-third of each) after blending the fine aggregates and silica fume, to improve the dispersion of the HRWR and to reduce the “wet-out” time of the mixture. If these liquid materials are added during the “dry-mixing” phase, additional mixing times may be required to break up balls of cementitious materials that may form when these materials come into contact with the pre-wetted aggregates and silica fume. The effectiveness of this process will likely vary depending on the admixtures used.
After all of the powder materials have been added, the water and chemical admixtures may be introduced. Water may be added all at once, or in increments. A portion (typically 5 to 10 percent) of the mixing water may also be retained as tail-water to be added at the end of the mixing process if sufficient flow has not been achieved. Chemical admixtures should be added as recommended by the manufacturer. Delayed addition of the HRWRs by one to two minutes usually results in more efficient dispersion and faster transition to a fluid state for the UHPC mixture.

The materials are then typically mixed until the UHPC achieves a fluid consistency. During this process, the mixture transitions from a dry powdery mixture, to a stiff plastic mixture, and finally to a flowable mixture. The transition from a stiff plastic mixture to a flowable mixture may require substantial torque from the mixer and is typically accompanied by a high current draw in the mixer motor. The mixer current may be monitored during trial batching to evaluate the demand that the given volume of materials is placing on the mixer motor, to verify that the intended volume of materials can be batched without damaging the mixer.

Fibers are usually added after the material has transitioned into a fluid mixture and the consistency of the material has been deemed satisfactory (e.g., by a flow spread test or visual assessment by experience technician). Fibers may also be added during the dry-mixing phase or concurrent with the water and chemical admixtures; however, fibers added early in the mixture may not achieve uniform dispersion or may result in balling with the materials, especially if only a small amount of moisture (e.g. from moisture content of aggregates) is present in the mixer. In all cases, fibers should be added using methods that promote uniform dispersion throughout the UHPC and prevent clumping. This may be accomplished via mechanical means or manual addition. Rapidly dumping fibers straight into the mixer from bags should be avoided, as this typically does not break up clumps of fibers already present within the bags and can result in clumps in the mix, as shown in Figure A.5.2.2-1. Adding fibers through gratings or with purpose-built fiber dispensing equipment that breaks up clumps and gradually adds the fibers can be beneficial (see Figure A.5.2.2-2). Appropriate personal protective equipment (PPE) should be used when distributing fibers manually.

After all of the materials have been added to the mixer and the UHPC has achieved a fluid consistency, mixing should continue until a uniform distribution of fibers has been achieved, then the batch may be discharged from the mixer.
Figure A.5.2.2-1. Clumps of Fibers in UHPC Caused by Too Rapid Addition of Fibers to the Mix
A.5.2.3. Moisture Control

Careful control of the water content of the mixture is essential to achieving consistent and satisfactory performance in both the fresh and hardened UHPC. The free moisture contribution from the aggregates should be properly accounted for in the batch quantities. If aggregates are used in a moist (i.e., not oven-dry) condition, accurate measurement of the aggregate moisture content is recommended for each batch. Moisture content may be determined by moisture probes in the aggregate bins and mixer, by rapid measurement techniques (e.g., ASTM D4944 / AASHTO T 217), or by oven-drying (e.g., ASTM C566 / AASHTO T 255). Correlations should be established between readings obtained via moisture probes or rapid measurement techniques and measurements obtained by oven-drying for each aggregate source prior to routine use in UHPC batching.

Measurement of aggregate moisture content by oven-drying or similar methods is recommended at least once per day.

Variability in initial aggregate moisture content, even when properly accounted for in the batch water, may result in variability in water demand, admixture demand, and initial flow, and may have additional consequences in terms of mechanical performance and durability. This is because the moisture variation may influence the effectiveness of the HRWR in dispersing the constituent materials during mixing.

A.5.2.4. Temperature Control

The temperature of fresh UHPC should be maintained between 50 and 80 °F (10 and 27 °C) during the batching and placement process. Obtaining these temperatures may require pre-heating or pre-cooling of the constituent materials or equipment, use of chilled water or ice, reduction in batch quantities to limit the heat of hydration of the cementitious materials, or batching and placement at particular times of the day to avoid extremes in ambient temperature. Fresh UHPC with temperatures approaching the upper end of this range may experience more rapid stiffening and loss of workability during placement, whereas fresh UHPC with temperatures near or below 50 °F (10 °C) may experience extended setting times and slower rates of strength gain. Some of these challenges may be overcome through the addition of chemical admixtures, such as...
accelerators or set retarders; however, maintaining the temperature of the UHPC within the target range has generally been an effective method of achieving consistent workability and setting times.

A.5.2.5. Batch Adjustments
At multiple points during mixing, it may be desirable to sample the mixture to determine if adequate flow has been achieved. This may be done just prior to fiber addition or after all of the fibers have been added, prior to discharge. If the mixture is deemed to have unsatisfactory flow, it can be either adjusted, mixed for additional time, or discarded, at the discretion of the batch plant operator and/or quality control personnel.

For mixtures with insufficient flow, batch adjustments may include addition of HRWR or water, provided the addition does not increase the total water content or admixture content of the mixture beyond acceptable levels. The water present in liquid admixtures will impact strength and durability and should be accounted for in the total water added to the mixture. Flow may also be improved with additional mixing time if it is believed that the cause of the insufficiency is inadequate dispersion of the mixing water or chemical admixtures. HRWR added later in the mixing process is likely to be much less efficient than the initial dose.

For mixtures with excessive flow, batch adjustments may include addition of more cementitious materials or a blend of pre-mixed powder. The cementitious materials should be added in similar proportions to the design mixture; however, it is cautioned that increasing the cementitious materials content of the material may increase the shrinkage of the UHPC. Excessive flow may also be remedied by delaying placement of the material until satisfactory flow is achieved; if flow is corrected in this way, the UHPC should be re-mixed prior to placement to ensure uniform distribution of the fibers at the time of placement.

The UHPC should be mixed after each adjustment until the added material is thoroughly incorporated so that the mixture is uniform. The adjusted mixture should be sampled and re-evaluated for flow. Any adjustment to the mixture proportions should be documented in the batch report. No adjustments to the mixture constituents should be made after discharge from the mixer.

A.5.2.6. Pre-Blended Materials
Producers may optionally wish to batch with a pre-blend of their local constituent materials to reduce production time in the mixer. Materials that are pre-blended typically include cement, silica fume, supplemental materials, and aggregates. Water, chemical admixtures, and fibers are typically added separately during batching. Chemical admixtures used in a powder form may be incorporated into the pre-blend, if desired; however, it is recommended that the amount of admixture added to the pre-blend is less than the target total batch proportion to permit adjustments during mixing. Fibers may also be incorporated into the pre-blend. If steel fibers are included in the pre-blend, it is possible that their relatively high density will cause them to settle at the bottom of the storage container. Thus, with pre-blends, it is important to agitate the dry constituents in the mixer before water and admixtures are introduced.

Pre-blending of constituent materials should be performed in a manner that results in a homogenous blend of the materials with minimal segregation during storage. All materials should be dry at the time of blending to avoid pre-hydration of the cementitious materials; this requires oven drying of the aggregates included in the mixture. Samples of pre-blended material should be taken periodically during production to ensure uniformity.

A.5.3. Handling and Placement

A.5.3.1. Transport
UHPC should be transported from the mixer to the placement location in a manner that avoids segregation, entrapment of air, contamination by foreign materials (e.g., residual concrete or stray aggregates), and dehydration of the UHPC material. Transport containers should be free of foreign material (e.g., conventional concrete, hardened concrete, debris) prior to introduction of the fresh UHPC to avoid contamination of the new material. It is advisable that the concrete be transported by mixing truck or a similar vessel capable of providing continuous, slow agitation of the material during transit to maintain the UHPC in a workable state and to limit segregation during transit and placement. If agitation cannot be provided during transit, transit and placement
times should be suitably short so as to maintain the workability of the UHPC and to limit settlement of the fibers between discharge from the batch plant mixer and placement in the forms.

It is generally recommended that the fresh UHPC be placed into the forms as soon after batching as possible. Like all concrete, UHPC will gradually lose its workability over time. Depending on the mixture components, UHPC has a typical target working time of 1 to 2 hours after addition of the mixing water; however, this may be extended through the use of chemical admixtures. The decrease in workability is faster for higher ambient temperatures, higher UHPC temperatures, and dry or windy conditions.

Producers are cautioned that a significant portion of the fresh UHPC will adhere to the surfaces of the batch plant mixer and the transport container. The amount of material retained is related to the viscosity of the UHPC mixture and will vary for each mix. The excess material retained on the mixer and transport container may be more than for typical conventional mixtures and should be accounted for in the batch quantities. Pressurized water or hand scrubbing may be required to completely remove excess material from the mixer or transport container during cleaning.

### A.5.3.2. Formwork

Formwork for UHPC can be designed with a variety of materials. Steel, glass, plywood, and various polymeric materials (e.g., silicone, polyurethane, epoxy-coated wood) have been successfully used for structural and architectural applications (National Precast Concrete Association, 2013). The self-consolidating behavior of many UHPCs permits the use of unique formwork shapes and surface textures that are difficult to be achieved with conventional concrete.

Formwork should be designed to withstand the full hydrostatic pressure of fresh UHPC. Hydrostatic pressure is calculated as the density of the UHPC, typically between 150 and 175 lb/ft³ (2400 to 2800 kg/m³), multiplied by the height of the formwork. Because the density of UHPC is greater than that of conventional concrete, formwork used for conventional concrete placements may not provide adequate support for UHPC pressures.

Forms should be grout-tight to prevent leakage of the UHPC after placement, and should be constructed to minimize the restraint of early-age volumetric changes of the fresh and setting UHPC. Considerable early-age (autogenous) shrinkage may occur due to the lack of coarse aggregates, the low water-cement ratio of the mixture, and the high dosage of silica fume and other supplemental materials used in the mix (see Section A.4.1.2). Thermal movement may also occur due to different heating and cooling rates of various portions of the element. Restraint of early-age shrinkage and thermal movement may be reduced by incorporating compressible materials into the formwork design, and by using hydrophobic, low-friction (smooth) surfaces combined with an appropriate form release agent, where possible. Stress concentrations due to shrinkage and thermal movement may be reduced by incorporating gradual transitions (e.g., chamfer or curved surfaces) between surfaces of differing angles or different cross-sections (Canadian Standards Association, 2019).

The maximum spacing between the faces of the formwork and any internal reinforcing or adjacent formwork should be no less than 1.5 times the fiber length or maximum aggregate size, whichever is greater, to permit adequate flow and consolidation of the UHPC (Canadian Standards Association, 2019).

### A.5.3.3. Placement

UHPC should be placed into the forms in a manner that prevents segregation, minimizes unfavorable alignment of the fibers, reduces the entrapment of air, and minimizes cold joints. In general, the UHPC should be placed as closely as possible to its final position, in a manner that integrates the new material into the previous layers of UHPC. This may be accomplished by continuously placing the UHPC from a single location via a “chimney” and using the UHPC’s self-consolidating properties or a confinement pressure to force the material outward; by injecting the UHPC into the bottom of the existing material via a tremie pipe; or by depositing the UHPC behind the leading edge of the flow so that it integrates with the flowing material. These placement methods are illustrated in Figure A.5.3.3-1. For casting of flexural members, placement by the first two methods is preferable to provide more favorable alignment of the fibers at the extreme ends of the beams; however, placement by the third method may be more practical depending on the product configuration and mixture performance. Free fall drops into the form from a height of greater than 3 feet (1 meter) above the placement
location are not recommended due to the increased potential for fiber segregation (Canadian Standards Association, 2019). Slower filling of the forms is preferable to reduce the entrapment of air.

Whenever possible, UHPC should be placed monolithically, in a single continuous pour. For example, locations where successive loads of UHPC overlap may become weak points in the element due to the lack of fibers bridging at the interface. Should such placements be necessary, placement procedures should ensure that the interface between adjacent pours acts monolithically. This may be accomplished by gently agitating the interface with a tamping rod (Japan Society of Civil Engineers, 2008) or with a slowly rotating propeller (Canadian Standards Association, 2019) immediately after placement to restore an isotropic fiber distribution at the interface, or by providing supplemental reinforcement at locations where placement may result in cold joints. Sequential lifts of UHPC should only be performed while the previous lift is still workable and must include provisions to restore fiber distributions across the interface where such factors are not accounted for in the structural design. Placing fresh UHPC onto a hardened, stiffened, or crusted surface will introduce a cold joint that may significantly limit the tensile performance of the element.

Because UHPC possesses self-consolidating properties, it will flow a certain horizontal distance in the forms from the point of placement. The horizontal flow distance is specific to each UHPC mixture, but may exceed 30 feet (9 meters) for highly flowable mixtures. As shown in Figure A.5.3.3-2, fibers will tend to orient along the direction of flow, so the longer the horizontal distance the UHPC flows, the more aligned the fibers will be at the extreme points of the flow. As described in detail in Section A.4.1.2, fiber alignment has a significant impact on the tensile performance of the UHPC. For this reason, placement should be planned to minimize unfavorable alignment of fibers in critical areas required by the structural design, such as the ends of flexural members. For girders with strands providing flexural capacity, but relying on UHPC’s tensile strength for vertical shear capacity at the ends of the beams, a centered, symmetric placement pattern (i.e., casting from a single location at midspan, two locations at third-points, etc.) is recommended to minimize unfavorable preferential fiber alignment on the end of the elements. Structural mock-ups evaluated based on input by the design engineer may be considered to verify that placement methods result in the intended distribution and alignment of fibers in the structural element (see Section A.6.5).
A.5.3.4. Consolidation
UHPC is self-consolidating and generally does not require appreciable external consolidation, except between sequential lifts, along sloped surfaces where voids may become trapped, or in tightly congested areas. Consolidation of UHPC should be conducted in a manner that does not generate fiber segregation or redistribution. Internal vibration should not typically be used, since internal vibrators will locally displace fibers where they are inserted. External vibration should only be used after verification that the intended consolidation methods do not adversely influence fiber distribution or alignment. If external vibration is used, it is recommended that vibration be applied in short intervals to limit segregation, particularly if lightweight synthetic fibers (e.g., PVA) which may float are used (National Precast Concrete Association, 2013). Extended periods of external vibration may cause segregation and/or misalignment of the fibers (Canadian Standards Association, 2019).

A.5.4. Finishing
Due to the fluid characteristics of the UHPC, its extremely low water content, and the dispersion of the fibers throughout the material, finishing of the surfaces by traditional means (e.g., screeding, raking, brooming) is typically not applicable. The self-consolidating and self-leveling properties of the UHPC may be relied on to achieve a smooth surface texture. Finishing with hand tools such as floats immediately after consolidation may be used if a flat, smooth finish is desired. Use of hand tools and other finishing implements should avoid tearing of the UHPC surface and disrupting the fiber orientation. Spiked rollers, as shown in Figure A.5.4-1, may be helpful for achieving a level surface without introducing surface tears.

If another layer is to be placed on top of the UHPC, a textured surface may need to be imparted to the interface surface to permit adequate bonding for composite action. Care should be taken to ensure that this texture is adequate and consistently achieved during production.
A.5.5. Curing
After finishing, all exposed surfaces should be immediately covered with plastic or wet burlap to prevent dehydration. Treatment with a curing compound may also be permitted, but the curing compound should be applied to the surface shortly after finishing is completed. The low water-cement ratio, coupled with the high silica fume dosage of the mixture, makes the fresh UHPC particularly susceptible to localized drying at the surface, especially when it is placed in high temperatures and/or low relative humidities. Within minutes, freshly-placed UHPC may begin to show a local drying and cracking phenomenon often referred to as “elephant skin” due to the resemblance of the rough, cracked surface to elephant’s skin, as shown in Figure A.5.5-1. Once an “elephant skin” appears on the UHPC’s surface, subsequent finishing is especially challenging. External vibration may break up some of the elephant skin that has formed and permit additional finishing of the surface, but the best way to avoid forming a skin on the surface of the UHPC is to limit the placement temperature of the UHPC and to finish and protect the surface as quickly as possible.
Figure A.5.5-1. “Elephant Skin” Formed on the Surface a UHPC Panel Element. The Skin Formed Due to Evaporation of Water from the Surface of the UHPC, and was Exacerbated by a Combination of High Concrete Temperatures at the Time of Placement, Warm Ambient Temperatures and Moderate Breeze, and a Large Exposed Surface Area

Structural UHPC elements are typically cured at ambient temperature until final setting is achieved. When cured at an ambient temperature of about 73 °F (23 °C), final set usually occurs within 24 hours of placement, but this may vary significantly depending on the mixture and the curing temperature. During curing, concrete temperatures may approach or exceed 160 °F (70 °C) due solely to the large heat of hydration of the cementitious materials. This may induce thermal stresses in the member, which may lead to cracking of the element if the element is restrained. Cooler concrete placement temperatures can help offset some of the temperature rise.

Early strength gain may be accelerated by providing supplemental heat to the element during curing, after final set; however, producers and designers are cautioned that accelerated curing prior to final set and while the product is still in the forms may significantly elevate the internal temperatures of the concrete member, and introduce considerable shrinkage and/or thermal effects that could cause cracking.

In accordance with the requirements presented in the design guidelines, the concrete should achieve a minimum compressive strength of 10,000 psi (70 MPa), or other specified level, prior to stripping and release of the prestressing force. This higher release strength compared to conventional concrete is required to support the higher prestressing forces required by the design. Compressive and flexural strength for prestressing release should be verified through laboratory testing of match-cured specimens cast at the same time as the structural element. The maturity method (ASTM C1074) may be used to estimate when prestressing release strength is achieved based on temperatures measured at a representative location within the UHPC element as it cures; however, strength at release must still be confirmed by compressive strength testing.
A.5.5.1. Thermal Treatment

After removal from the forms, a thermal treatment (secondary heat cure) may optionally be applied to the element. Thermal treatment is not required, but may increase strength and improve long-term durability, volumetric stability, and fire resistance. UHPC that has been thermally treated has been shown to have higher strengths, lower permeability to moisture, and decreased long-term shrinkage and creep after thermal treatment compared to UHPC cured at ambient temperatures, allowing for improved dimensional stability and reduced prestressing losses over time (Canadian Standards Association, 2019; ACI Committee 239, 2018; Garas, Kahn, and Kurtis, 2009; Swiss Society of Engineers and Architects (SIA), 2016). Thermal treatment has also been noted to improve fire resistance by removing excess water from the concrete’s pores, thereby reducing the potential for spalling (Canadian Standards Association, 2019). For certain UHPC formulations, thermal treatment may be required to activate all of the cementitious materials to achieve the full potential strength and performance characteristics (Khayat and Valipour, 2018; Russell and Graybeal, 2013).

A commonly specified thermal treatment procedure for UHPC containing steel fibers is to provide steam at 195 ± 5 °F (90 ± 3 °C) for 48 hours within the first 14 days of placement (Canadian Standards Association, 2019; National Precast Concrete Association, 2013). Other thermal treatment options, such as simple steam curing, or heating to lower temperatures or for different durations may be used, provided that the UHPC consistently achieves the desired mechanical and durability properties following the thermal treatment (see Section A.4). If it can be demonstrated that the intended mechanical and durability properties may be achieved without a secondary thermal treatment, then this procedure may be eliminated.

Thermal treatment, if used, should be applied in a controlled manner so that the rate of temperature increase or decrease in the element does not exceed 20 °F (10 °C) per hour (Canadian Standards Association, 2019). Sufficient moisture should be provided to ensure that the concrete surface does not dry out during treatment. Thermal treatment is commonly applied shortly after removal of the UHPC element from the forms, but may be applied at any time within the first approximately 14 days of placement (Alsalman, Dang, and Hale, 2017). For conventional concrete mixtures, the potential for delayed ettringite formation (DEF) is usually considered if curing temperatures exceed 158 °F (70 °C); however, with the low porosity and high silica fume contents, DEF is generally not a concern for UHPC even if thermal treatment is applied (see Section A.4.1.3).

Note, curing UHPC at temperatures of 195 °F (90 °C) may cause non-metallic fibers to melt.
A.6. Quality Inspection and Testing

Consistent production of high-quality UHPC elements requires regular quality inspection and testing at each stage of the production process. Many quality inspection protocols employed for conventional precast concrete production are applicable to UHPC. The following sections describe quality inspection and testing requirements that differ from conventional concrete production and therefore should be incorporated into the plant’s quality control program for UHPC production.

A.6.1. Materials

The constituent materials used for UHPC production should meet all applicable ASTM, AASHTO, and other governing materials specifications. Manufacturers' mill certificates and test reports should be reviewed for each shipment to verify compliance with applicable standards and any additional chemical or physical requirements specified by the UHPC mix design. When necessary, additional testing should be performed in accordance with the relevant ASTM, AASHTO or other standards, as described in Chapter A.4.

Aggregate moisture content should be accurately measured and recorded for each batch, as described in Section A.5.2.3. Mixture proportions should be adjusted as appropriate to account for free water provided by the aggregates and by the chemical admixtures (see Section A.3.2.3).

A.6.2. Batching

Constituent materials should be weighed on scales accurate to at least 2 pounds (1 kg), or at least 1 percent of the batch weight of material, whichever is smaller. Material weights should be recorded for each batch and compared to permissible tolerances. If no tolerances are specified, the following tolerances are recommended (Canadian Standards Association, 2019; Japan Society of Civil Engineers, 2008; Precast/Prestressed Concrete Institute, 1999):

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum Batching Error (%)</th>
<th>Maximum Batching Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>With Plant Batched Materials</td>
<td>With Pre-blended Dry Materials</td>
</tr>
<tr>
<td>Water</td>
<td>± 1%</td>
<td>± 1%</td>
</tr>
<tr>
<td>Cement</td>
<td>± 1%*</td>
<td>± 2% (total pre-blend)</td>
</tr>
<tr>
<td>Silica Fume</td>
<td>± 1%*</td>
<td></td>
</tr>
<tr>
<td>Supplemental Material (SCM or other inert filler)</td>
<td>± 1%*, or ± 5 lb, whichever is greater</td>
<td></td>
</tr>
<tr>
<td>Aggregate</td>
<td>± 2%*</td>
<td></td>
</tr>
<tr>
<td>Fiber</td>
<td>-2%, +4%</td>
<td>-2%, +4%</td>
</tr>
<tr>
<td>Chemical Admixture</td>
<td>± 2%</td>
<td>± 2%</td>
</tr>
</tbody>
</table>

* Total weight of dry materials should not exceed ± 2% of target.

If materials are added from pre-weighed bags, the bags should be weighed at a frequency of one per every ten bags used to verify that batching tolerances are met (Precast/Prestressed Concrete Institute, 1999).

A mixing protocol should be established and documented for each concrete mixture as part of the mixture qualification process (see Section A.4). This protocol should include, at minimum, the sequence of materials addition and the target times for each step. Any variations from the qualified mixing protocol should be approved by the design engineer or an experienced UHPC technician and documented. Additional mixture qualification testing may be necessary if the mixing protocol is significantly altered. At minimum, the time at the start of mixing, the time at water addition, the time of fiber addition, and the time at discharge should be recorded.

If pre-blended materials are used, a sample of pre-blended material should be retained for the first batch of pre-blend and every tenth batch thereafter for each production run, in sufficient quantity that the requisite fresh and hardened properties of the material (described below) may be evaluated if needed (Association Francaise de Normalisation (AFNOR), 2016).
A.6.3. Plastic Testing

Batching procedures should be validated from samples taken at the time of discharge. The fresh (plastic) UHPC properties to be evaluated include the total flow spread, temperature, and density (Table A.6.3-1). Flow spread and temperature should be evaluated according to ASTM C1856 and ASTM C1064, respectively, at a minimum frequency of once per batch. While C1856 indicates a brass flow table (ASTM C230) is to be used, a brass table is generally not essential and may be too small; provided that the UHPC is spread onto a clean, smooth, flat, and level surface that will not absorb water from the fresh UHPC, the brass table requirement may be omitted. It may be desirable to perform additional flow and temperature testing at intermediate stages of the batching process, such as immediately prior to the addition of fibers. UHPC batches failing to meet the specified flow or temperature requirements should be either adjusted as described in Section A.5.2.5 or rejected.

Density (unit weight) should be measured in accordance with a modified ASTM C138 for at least the first batch per day. Additional testing is recommended any time test specimens are produced, and whenever a change in mix quality is suspected. Equipment used to measure the density of conventional concrete may also be used for UHPC, provided that the UHPC is poured into the measure in a single lift, and the measure is tapped 30 times with a rubber mallet to provide adequate consolidation. Density is used as an indicator of entrapped or entrained air content, deviations in batching weights, or fiber segregation, all of which may negatively impact the performance of the hardened UHPC. Mixtures with measured densities significantly below the theoretical density of the mixture may produce decreases in compressive strength and/or durability and should not be used without approval of the design engineer or experienced UHPC technician.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Minimum Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow</td>
<td>ASTM C1856</td>
<td>Each batch</td>
</tr>
<tr>
<td>Temperature</td>
<td>ASTM C1064</td>
<td>Each batch</td>
</tr>
<tr>
<td>Density</td>
<td>ASTM C138, modified</td>
<td>First batch per day, whenever test specimens are cast, and whenever a change in quality is suspected</td>
</tr>
</tbody>
</table>

A.6.4. Hardened Testing

The hardened properties used to validate the batching process include compressive and flexural strength (Table A.6.4-1). Compressive strength should be evaluated for each mixture according to ASTM C39, as modified by ASTM C1856. Testing should be performed for at least three 3-inch by 6-inch cylindrical specimens at each age tested, and should include at minimum, verification of the strength at prestress release and verification of the strength at 28 days (or at service) for each day’s production. Cylinders should be fabricated in a single lift and consolidated by tapping the sides of the molds as specified in ASTM C1856.

If possible, test specimens (especially cylinders) used for verification of strength at prestress release should be match-cured with the concrete element to provide a more accurate measurement of the compressive strength, which is strongly influenced by the temperature of the concrete during curing. Match curing should be performed according to AASHTO R72, Standard Practice for Match Curing of Concrete Test Specimens, except that the specimen sizes, specimen molding, and other testing are to be performed according to practices outlined in this document not AASHTO R72. Specimens not used for verification at prestress release can be match-cured or cured according to ASTM C1856. The curing procedure(s) used for each set of test specimens should be documented.

There are some important differences in compressive strength testing procedures for UHPC per ASTM C1856 that should be recognized: (1) compressive strength of UHPC is evaluated using smaller, 3-inch by 6-inch cylinders to limit the required load capacity of the testing machine; (2) the ends of the cylinders are ground plane prior to loading, rather than using bonded or unbonded caps, to ensure that the measured strength is not affected by surface irregularities or capping material strength limitations or defects; and (3) a faster load rate

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4 Testing at standard milestone ages, such as 28 or 56 days is customary; however, testing at service may also be performed if specified.
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of 145 psi (1 MPa) per second is used to reduce the total duration of the test. Compressive strength testing should be performed by a qualified laboratory technician using the modifications described in ASTM C1856. Failure to grind the cylinder surfaces to the specified planeness may result in unrepresentative strength results, and should be avoided. Testing with bonded or unbonded caps may be useful at early ages to provide confirmation that a minimum strength has been achieved, but should not be performed once the concrete strength has exceeded 12,000 psi (83 MPa) due to the potential for unrepresentative strengths to be measured (see Section A.4.1.2).

Because the relationships between the compressive strength of a UHPC and its flexural properties will vary depending on the mix, flexural strength should also be measured as part of routine quality control testing. Flexural strength should be measured under four-point bending according to ASTM C1609 (as modified by ASTM C1856). While some precast concrete producers may already perform four-point flexural strength testing routinely on conventional concrete, the testing is likely performed according to ASTM C78, which has different requirements for the testing apparatus, load application, and data collection and analysis. Equipment used for ASTM C78 testing should be checked for compliance before using it for testing under ASTM C1609. It is important that the testing apparatus have freely-rotating supports to permit rotation of the specimen after cracking, and that the loading frame be equipped with a servo control mechanism to control the rate of loading after the first crack has initiated. Restraint of specimen rotation by the supports may erroneously increase the measured flexural strength by up to 40 percent (Wille and Parra-Montesinos, 2012), while failure to control the load rate after first cracking may prevent accurate measurement of the peak (ultimate) flexural strength and the residual flexural strength of the material.

Flexural strength specimens should be cast according to ASTM C1856, by pouring continuously from one end of the mold. Testing should be performed at 28 days (or at service) to verify that the design flexural strength has been achieved. First-peak (first-crack) strength should also be evaluated at least once per production run at the time of prestress release to verify that sufficient tensile capacity is provided by the UHPC. At the present time, it may be necessary for many precasters to have the flexural performance of UHPC evaluated by an external testing laboratory. As more research is conducted, new testing protocols may be developed that can be more readily implemented at the plant level.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Minimum Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength</td>
<td>ASTM C1856</td>
<td>6 cylinders from one batch per day or every 25 cubic yards (20 cubic meters), whichever is more frequent; test 3 cylinders at prestress release and 3 cylinders at 28 days (or at service)</td>
</tr>
<tr>
<td>First-Peak (First-Crack)</td>
<td></td>
<td></td>
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<tr>
<td>Flexural Strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peak (Ultimate) Flexural</td>
<td>ASTM C1856</td>
<td>3 beams from one batch per day or every 25 cubic yards (20 cubic meters), whichever is more frequent; test 3 beams at 28 days (or at service)</td>
</tr>
<tr>
<td>Strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Residual Flexural Strength</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A.6.5. Product Testing

Routine laboratory quality control testing can provide valuable information about the UHPC’s performance and characteristics as batched; however, it does not capture any effects on the material’s performance that may result from fabrication processes as the UHPC is placed within a structural element. Factors related to placement and handling such as the alignment of fibers within an element, the flow of the fresh UHPC material around corners and voids, the bridging of fibers between adjacent or successive lifts, and the ability of the UHPC to consolidate around dense reinforcing steel without the aid of internal vibration, all have a significant impact on the overall performance of an element that cannot be quantified through standard quality control testing procedures.
Because UHPC is currently considered an emerging material, some owners and specifiers may require the production of a full-scale mock-up element to demonstrate that the element can be successfully produced using a particular plant’s materials, batching procedures, and production methods. Proof testing may also be specified to verify that the element can achieve the minimum design requirements with respect to strength and ductility under design and service loads.

In cases where fiber alignment, material consolidation, placement sequencing, or other aspects of production may have a critical impact on the structural performance of a precast UHPC element, production of mock-up elements may also be advisable, even if they are not required by the owner or specifier. In such cases, the mock-up element should be produced using the same materials, batching sequence, handling, and placement procedures anticipated for full production, but does not necessarily need to be a full-size element. The element should be adequately sized so that the particular effect(s) of concern can be evaluated. For example, if there is concern about the ability of the material to flow through a narrow opening around a void in a long-span element, a mock-up element measuring only a few feet in length may be sufficient for evaluation.

Testing of the mock-up element will vary based on the intent of its production. For example, if the purpose of the mock-up is to understand if the material can flow sufficiently through a complex form, it may be sufficient to cut a cross-section of the element to visually examine consolidation and voiding. If the purpose is to better understand the impact of fiber alignment on the overall performance of an element, samples may be saw-cut from representative locations within the element and visually examined for fiber alignment. If it is desirable to understand the impact of placement methods and fiber alignment on the flexural performance of a beam element, flexural test specimens may be saw-cut from representative locations along the tension face of the element and evaluated for flexural performance along the longitudinal and transverse axes. In any case, the mock-up element should be designed so that the effect(s) of concern may be properly evaluated using the evaluation techniques established by the designer, production lead, or quality control lead.
A.7. References

Authored References

ACI Committee 201. (2016). *ACI 201.2R-16, Guide to Durable Concrete*. Farmington Hills, MI: American Concrete Institute.


Japan Society of Civil Engineers. (2008). *Concrete Engineering Series 82, Recommendations for Design and Construction of High Performance Fiber Reinforced Cement Composites with Multiple Fine Cracks (HPFRCC)*. Japan: Japan Society of Civil Engineers.


Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix A – Draft Guidelines for Production


Referenced ASTM Standards

ASTM A820 / A820M - 16, Standard Specification for Steel Fibers for Fiber-Reinforced Concrete
ASTM C31 / C31M - 18b, Standard Practice for Making and Curing Concrete Test Specimens in the Field
ASTM C33 / C33M - 18, Standard Specification for Concrete Aggregates
ASTM C39 / C39M - 18, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
ASTM C78 / C78M - 18, Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
ASTM C138 / C138M - 17a, Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete
ASTM C144 - 18, Standard Specification for Aggregate for Masonry Mortar
ASTM C150 / C150M - 18, Standard Specification for Portland Cement
ASTM C157 / C157M - 17, Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
ASTM C469 / C469M - 14, Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression
ASTM C494 / C494M - 17, Standard Specification for Chemical Admixtures for Concrete
ASTM C512 / C512M - 15, Standard Test Method for Creep of Concrete in Compression
ASTM C566 - 13, Standard Test Method for Total Evaporable Moisture Content of Aggregate by Drying
ASTM C618 - 17a, Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
ASTM C642 - 13, Standard Test Method for Density, Absorption, and Voids in Hardened Concrete
ASTM C666 / C666M - 15, Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
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Appendix A – Draft Guidelines for Production

ASTM C989 / C989M - 18, Standard Specification for Slag Cement for Use in Concrete and Mortars

ASTM C1012 / C1012M - 18b, Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution

ASTM C1017 / C1017M - 13e1, Standard Specification for Chemical Admixtures for Use in Producing Flowing Concrete

ASTM C1064 / C1064M - 17, Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete

ASTM C1074 - 17, Standard Practice for Estimating Concrete Strength by the Maturity Method

ASTM C1202 - 18, Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

ASTM C1218 / C1218M - 17, Standard Test Method for Water-Soluble Chloride in Mortar and Concrete


ASTM C1260 - 14, Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)

ASTM C1293 - 18, Standard Test Method for Determination of Length Change of Concrete Due to Alkali-Silica Reaction


ASTM C1556 - 11a (2016), Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion

ASTM C1609 / C1609M - 12, Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)

ASTM C1610 / C1610M - 17, Standard Test Method for Static Segregation of Self-Consolidating Concrete Using Column Technique

ASTM C1611 / C1611M - 18, Standard Test Method for Slump Flow of Self-Consolidating Concrete


ASTM C1778 - 16, Standard Guide for Reducing the Risk of Deleterious AlkaliAggregate Reaction in Concrete

ASTM C1797 - 17, Standard Specification for Ground Calcium Carbonate and Aggregate Mineral Fillers for use in Hydraulic Cement Concrete

ASTM C1856 / C1856M-17, Standard Practice for Fabricating and Testing Specimens of Ultra-High Performance Concrete

ASTM D4944 - 18, Standard Test Method for Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester

Referenced AASHTO Standards


AASHTO R 72 - 16, Standard Practice for Match Curing of Concrete Test Specimens


AASHTO T 255 - 00 (2017), Standard Method of Test for Total Evaporable Moisture Content of Aggregate by Drying (ASTM C 566-97 (2004))

AASHTO T 259 - 02 (2017), Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration

AASHTO T 277 - 15, Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (ASTM C 1202-12)
B. APPENDIX B - DRAFT MATERIALS GUIDE SPECIFICATION

Preface

This Guide Specification is intended to be used as a basis for the preparation of specifications for a specific application. This Guide Specification must be edited to fit the conditions of use. Particular attention should be given to the deletion of inapplicable provisions or inclusion of appropriate requirements. Coordinate the specifications with the information shown on the contract drawings to avoid duplication or conflicts.

This Guide Specification has been developed for use in connection with the structural design of precast pretensioned concrete beams for both building and transportation structures. A typical specification based on the MasterFormat (as may be used for AIA or NAVFAC structures) may consist of three parts: Part 1 - General, Part 2 - Products, and Part 3 - Execution. A transportation agency may have separate related documents: a UHPC Materials Specification and a Precast UHPC Element Fabrication Specification. Each of these transportation agency documents will typically consist of three or more parts, most commonly: Section 1 - Description, Section 2 - Materials, and Section 3 - Construction. It is intended that the sections of this Guide Specification may be inserted into the applicable sections of either specification type, as recommended in the table below.

This Guide Specification addresses the materials and production of structural precast, prestressed ultra-high performance concrete (UHPC) beams. It does not address structural design or erection requirements, and excludes the following: structural performance requirements, connection materials and details, anchorage and other hardware requirements, fire-resistance ratings, reinforcing materials (except fibers) and reinforcement fabrication, welding, finishes, delivery, storage and handling of UHPC members, erection, and cleaning. Further guidance on those topics is available in the PCI Guide Specification: Structural Precast and in the structural design guidelines for UHPC.
### Appendix B – Draft Materials Guide Specification

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<th>Section in this UHPC Materials Guide Specification</th>
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<th>Recommended Transportation Agency Specification Section</th>
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<td>B.1 Scope</td>
<td>Part 1 - General: Summary</td>
<td>Precast UHPC Element Fabrication Specification Section 1 - Description; revise as necessary to suit scope of individual document.</td>
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<td>B.2 Terms and Definitions</td>
<td>May be added as part of Part 1 - General</td>
<td>Relevant terms may be included in UHPC Materials Specification Section 1 - Description OR in Precast UHPC Element Fabrication Specification Section 1 - Description, OR entire list may be included in overall Project Specifications.</td>
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<tr>
<td>B.3 General</td>
<td>Part 1 - General: Submittals</td>
<td>Material Identity Card and Material Test Reports to be included in UHPC Materials Specification Section 2 - Materials. Material Identity Card and Field Quality Control Tests to be included in Precast UHPC Element Fabrication Specification Section 3 - Construction.</td>
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<td>B.3.1 Submittals</td>
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<td>B.4.1 Form Materials</td>
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<td>B.4.2 UHPC Materials</td>
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<td>B.4.3 UHPC Mixtures</td>
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<td>B.5 Fabrication</td>
<td>Part 2 - Products: Fabrication</td>
<td>Precast UHPC Element Fabrication Specification Section 3 - Construction UHPC Materials Specification Section 3 - Construction</td>
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<td>B.5.2 Concrete</td>
<td>Part 2 - Products: Concrete</td>
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<tr>
<td>B.5.3 Production Mock-Up</td>
<td>Part 2 - Products: Source Control</td>
<td>Precast UHPC Element Fabrication Specification Section 3 - Construction</td>
</tr>
</tbody>
</table>
B.1. Scope
   A. This Section applies to ultra-high-performance concrete (UHPC) intended for use in structural precast, pretensioned bridge and building beam elements. The work performed under this Section includes all labor, material, equipment, related services, and supervision required for the manufacture of the structural precast and precast, prestressed UHPC work shown on the Contract Drawings.

B.2. Terms and Definitions
   A. The following definitions apply to terms in this Section.
      1. **Acceptance Testing** - Testing performed during production of a UHPC member to verify that the UHPC complies with Project specifications. Acceptance testing considers the specific lot of materials, batch proportions, batching procedures, and production methods used to produce each UHPC member.
      2. **Batch** - A volume of materials placed into a mixer and uniformly blended, then discharged.
      3. **Binder** - The combination of hydraulic cement, supplementary cementitious materials, and mineral fillers included in a UHPC mixture.
      4. **Flexural Strength** - The maximum tensile stress value sustained by a test specimen due to bending.
         a. **First-peak (first crack) flexural strength** - The stress value obtained at the first point on the flexural load-deflection curve where the slope is zero; this typically corresponds to the stress value at the time of first cracking.
         b. **Peak (ultimate) flexural strength** - The maximum engineering stress value obtained in a flexural test, computed using simple bending theory for linear elastic materials and gross (uncracked) section properties.
         c. **Residual flexural strength** - The engineering stress value obtained, after cracking, at a specified net deflection (e.g. L/150, where L is span length in test) in a flexural test. Residual flexural strength is computed using simple bending theory for linear elastic materials and gross (uncracked) section properties.
      5. **Flow Spread** - the distance of lateral flow of UHPC during a flow test conducted in accordance with ASTM C1856.
      6. **Informational Testing** - Testing that is not required by Project specifications performed to obtain additional information about material properties or durability characteristics of the UHPC.
7. **Material Identity Card** - A document that provides the details of the constituent materials, mixture proportions, mixing instructions, curing methods (including thermal treatment, if used), and resulting hardened properties of a specified UHPC mixture.

8. **Mineral Filler** - A finely-divided, inorganic material, usually derived from quarried stone that may be used in UHPC to improve certain properties.

9. **Pre-Blend** - A uniform mixture of powder constituents into which water and admixtures are added during batching. Fibers may or may not be included in the pre-blend.

10. **Qualification Testing** - Testing performed prior to production to characterize the performance of a UHPC mixture and to demonstrate that the material can meet all Project requirements. Qualification testing considers the mixture components, proportions, mixing procedures, curing, and thermal treatment intended to be used for the Project.

11. **Thermal Treatment** - A process of heating the UHPC to an elevated temperature, above normal heat of hydration, in the presence of high relative humidity, holding the elevated temperature for a period of time to promote the hydration process, and then slowly cooling to ambient temperature.

12. **Total Water** - The combination of the batched water and ice, free moisture added from (or removed by) the aggregates, and the liquid water portion of all chemical admixtures and silica fume slurry.

13. **Ultra-High-Performance Concrete (UHPC)** - A fiber-reinforced, cementitious material characterized by highly refined microstructure, post-cracking ductility, high tensile strength, high compressive strength and excellent durability properties.

14. **Water-Binder Ratio** - The weight of total water in a UHPC mixture divided by the weight of total binder.

15. **Working Time** - The period of time after the introduction of water during mixing, over which a UHPC mixture maintains a flow spread of at least 8 inches (200 mm). Working time may be evaluated by sampling fresh UHPC at 15-minute increments until two successive flow spread measurements of less than 8 inches (200 mm) are recorded. The working time is the time at which the flow spread equals 8 inches (200 mm), as determined by linear interpolation. Working time will vary with ambient conditions, concrete temperature, mixing procedures, and transport methods.

## B.3. General

### B.3.1. Submittals

Retain test reports paragraph below if submittal is required. Edit list to suit Project.

#### B.3.1.1. Material Identity Card

A. Submit a Material Identity Card for each UHPC mixture to be used in the Project. The properties listed in the Material Identity Card shall demonstrate, at minimum, compliance with all Project material property requirements.

B. The Material Identity Card shall include the following:

1. The type and source of each constituent material.
2. The mixture proportions.
3. Mixing procedures.
4. Curing and thermal treatment (if used) procedures.

Revise Subsection references in paragraph below as needed.

5. The properties of the UHPC in accordance with Subsections B.4.3.3, B.4.3.4, and B.4.3.5.
Implementation of UHPC in Long-Span Precast Pretensioned Elements


a. Reported properties shall include, at minimum, all properties indicated in Table B.4.3.5-1 as “Qualification” and “Informational - Mandatory” tests.

b. Additional properties and performance characteristics listed as “Informational - Optional” in Table B.4.3.5-1 may be reported for informational purposes. “Informational - Optional” test results may be included in the Project submittals but are not required for approval.

c. Include the date of testing; the temperature, flow spread, and unit weight of each batch produced for Qualification purposes; the number of tests; the mean result of all tests; the standard deviation of results for each test; and the name, address, and contact information of the laboratory where the testing was performed.

B.3.1.2. Strength Test Records

A. Submit documentation indicating that the proposed UHPC proportions will produce average compressive and flexural strengths equal to or greater than the required average compressive and flexural strengths, respectively.

1. The minimum required average compressive and flexural strengths shall be determined as defined in Subsection B.4.3.5 based on the standard deviation of test results and the specified strengths defined in Subsection B.4.3.3.

B. Documentation shall consist of strength test records as defined in Subsection B.4.3.5.

B.3.1.3. Material Test Reports

A. Mill or suppliers’ test certificates or compliance test reports shall be submitted for the following materials:

1. Cementitious materials.
3. Aggregates.
5. Chemical admixtures.
6. Fibers.
7. Other components specified in Contract Documents with applicable standards.

B.3.1.4. Quality Control Test Reports

A. Submit quality-control test reports as described in Section B.5.2.7.B.

B.4. Materials

B.4.1. Form Materials

A. Form materials: Rigid, dimensionally stable, non-absorptive material, warp and buckle free, that will provide precast UHPC surfaces within fabrication tolerances indicated; nonreactive with concrete and suitable for producing required surface finishes.

B. Form-Release Agent: Commercially produced form-release agent that will not bond with, stain or affect hardening of precast UHPC surfaces and will not impair subsequent surface or joint treatments of precast UHPC.

B.4.2. UHPC Materials

A. Use the same type, brand, and mill source for all materials throughout the UHPC production. Changes in the type or source of cementitious materials, mineral fillers, aggregates, admixtures, or fibers require the submittal and approval of an updated Material Identify Card (see Section B.3.1.1) for the modified UHPC mixture produced with the alternative material type(s) or source(s).

Retain materials in this article that are required. Revise to suit Project.
Implementation of UHPC in Long-Span Precast Pretensioned Elements


B. Hydraulic Cement: ASTM C150, ASTM C595, or API SPEC 10A.

C. Supplementary Cementitious Materials and Mineral Fillers:

Consult local fabricators prior to selecting mineral or cementitious materials from the six sub-paragraphs below. These materials may affect UHPC appearance, workability, setting times, strength, and cost.

1. Fly Ash: ASTM C618, Class C or F.
2. Raw or Calcined Natural Pozzolan: ASTM C618, Class N.
5. Ground Calcium Carbonate and Aggregate Mineral Fillers: ASTM C1797.
6. Other Supplementary Cementitious Materials or Mineral Fillers: Demonstrate suitable mechanical and durability performance when used in UHPC. Suitability to be determined at the discretion of the Engineer.

D. Fine Aggregates: ASTM C33 or ASTM C144

1. Fine aggregates do not need to comply with the grading requirements in ASTM C33 or C144.
2. Alkali-silica reactivity requirements may be waived if silica fume content is equal to or greater than 8 percent of the total weight of the binders. Otherwise, aggregates that are classified as highly or very highly alkali-silica reactive by ASTM C1778 shall not be used.
3. Aggregates that are potentially expansive by alkali-carbonate reaction as defined by ASTM C1778 shall not be used.

E. Water: Potable municipal water or other water complying with chemical limits of PCI MNL 116 or ASTM C1602.

F. Chemical Admixtures: Certified by manufacturer to be compatible with other admixtures and to not contain calcium chloride, or more than 0.15 percent chloride ions by weight of admixture.

1. Water-Reducing Admixture: ASTM C494/C494M, Type A.
2. Retarding Admixture: ASTM C494/C494M, Type B.
3. Water-Reducing and Retarding Admixture: ASTM C494/C494M, Type D.
4. Water-Reducing and Accelerating Admixture: ASTM C494/C494M, Type E.
5. High Range, Water-Reducing Admixture: ASTM C494/C494M, Type F.
6. High Range, Water-Reducing and Retarding Admixture: ASTM C494/C494M, Type G.
7. Viscosity or Rheology Modifying Admixture: ASTM C494/C494M, Type S.
8. Shrinkage Reducing Admixture: ASTM C494/C494M, Type S.
9. Plasticizing Admixture for Flowable Concrete: ASTM C1017/C1017M.

G. Steel Fibers: ASTM A820/A820M, with a minimum tensile strength of 250,000 psi (1,700 MPa) and a minimum nominal aspect ratio of 60, unless otherwise approved by the Engineer based on demonstrated adequate performance when used in UHPC.

Retain the paragraph below if non-metallic fibers are permitted. Revise list to suit Project. Polypropylene (PP) or polyvinyl alcohol (PVA) fibers may be used if fire resistance is required. The dosage of non-metallic fibers used must be sufficient to provide fire resistance complying with applicable requirements.

H. Non-Metallic Fibers:

1. Polypropylene Fibers: ASTM C1116/C1116M for Type III Synthetic Fiber-Reinforced Concrete.
2. Polyvinyl Alcohol Fibers: ASTM C1116/C1116M for Type III Synthetic Fiber-Reinforced Concrete.

Retain the paragraph below if pre-blended UHPC materials are permitted.

I. Pre-blended UHPC materials, for which Material Identify Card has been approved by Engineer.
   1. Follow all Manufacturer’s recommendations for handling, mixing, and placing.
   2. Retain a sample of the pre-blended material from each lot received, in sufficient quantity that the quality control testing of the fresh and hardened properties of the material may be evaluated for compliance.

B.4.3. UHPC Mixtures

B.4.3.1. Mixture Proportions
   A. Proportion mixtures with materials to be used on Project to provide UHPC with the following characteristics:

      A water-binder ratio of 0.20 or less is typical for UHPC production.
      1. Maximum water-binder ratio (w/b): 0.21
      2. Minimum steel fiber content: 2% steel fibers, by volume of concrete (263 lb./yd³).

Retain the first paragraph below if fire resistance is required.

3. Minimum non-metallic fiber content: as needed to achieve specified fire resistance.
4. Maximum water-soluble chloride ions: ASTM C1218/C1218M, 0.06% by weight of cement.
5. Minimum fresh properties as defined in Subsection B.4.3.2.
6. Minimum hardened properties as defined in Subsection B.4.3.3.

Retain the paragraph below if durability requirements are specified.

7. Minimum durability properties as defined in Subsection B.4.3.4.

B.4.3.2. Requirements for Fresh UHPC
   A. The fresh UHPC shall have the following properties:
      1. Temperature: ASTM C1064, between 50 °F (10 °C) and 80 °F (27 °C) at the time of placement, unless otherwise approved by the Engineer.

Revise the paragraph below if alternative flow spread limits are required. A flow spread of 11 inches (275 mm) may be associated with segregation or settlement of steel fibers. Potential for fiber settlement should be evaluated in accordance with Section B.5.2.3.D.

   2. Flow Spread: ASTM C1856, 8 to 11 inches (200 to 275 mm), measured not longer than 15 minutes before anticipated placement time.
   3. Working time sufficient for element fabrication.

B.4.3.3. Requirements for Hardened UHPC
   A. The hardened UHPC shall have the following properties:
      1. Compressive Strength: ASTM C1856, with the exception that either 3- or 4-inch diameter specimens may be used
         a. Release: 10,000 psi (69 MPa), minimum.
         b. At service: 18,000 psi (124 MPa), minimum. Age at service may be defined as 28 days, 56 days, or another period as indicated by the Engineer.
      2. Flexural Performance: ASTM C1856, at service:
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- First-peak (first crack) flexural strength: 1,500 psi (10.3 MPa), minimum.
- Peak (ultimate) flexural strength: 2,000 psi (13.8 MPa), minimum.
- Peak (ultimate) shall be at least 1.25 times first-peak (first crack) flexural strength.
- Residual flexural strength at midspan deflection of L/300: 90% of first-peak flexural strength, minimum.
- Residual flexural strength at midspan deflection of L/150: 75% of first-peak flexural strength, minimum.
- Age at service may be defined as 28 days, or 56 days, or another period as indicated by the Engineer of Record.

Retain the applicable subsections below if durability criteria are specified.

B.4.3.4. Durability Requirements for Hardened UHPC

A. The hardened UHPC shall exhibit the following durability characteristics:

1. Indication of Resistance to Chloride Ion Penetration (28 days): ASTM C1856, 500 coulombs, maximum.
   a. If the UHPC mixture design contains metallic fibers, testing shall be performed on specimens produced from UHPC having the same relative proportions of the other constituent materials but produced without metallic fibers.

2. Chloride Diffusion Coefficient (28 days): ASTM C1556, 0.025 in²/yr (0.5×10⁻¹² m²/s)
   a. If the UHPC mixture design contains metallic fibers, testing shall be performed on specimens produced from UHPC having the same relative proportions of the other constituent materials but produced without metallic fibers.
   b. Expose specimens to test solution for 90 days.

3. Sulfate Resistance: ASTM C1012
   a. If the UHPC mixture design contains metallic or non-metallic fibers, testing shall be performed on specimens produced from UHPC having the same relative proportions of the other constituent materials but produced without fibers.
   b. Expose specimens to test solution at 28 days.

4. Resistance to Freezing and Thawing: ASTM C1856, minimum relative dynamic modulus of 95 after 300 cycles

5. Absorption (28 days): ASTM C642, 3.0% maximum

B.4.3.5. Mixture Qualification

A. Material testing of each UHPC mixture shall be performed to support preparation of a Material Identity Card and Strength Test Records, as defined in Subsections B.3.1.1 and B.3.1.2, respectively, for the purpose of mixture qualification.
B. Testing for mixture qualification shall include, at minimum, the properties listed as “Qualification” and “Informational - Mandatory” in Table B.4.3.5-1.

1. All test specimens shall be fabricated from batches of UHPC produced with the same materials, mixture proportions, batching equipment, and mixing sequence intended for the Project.

2. The temperature and flow spread of the UHPC shall be within the required ranges at the time of specimen fabrication.

3. Document the temperature, flow spread, and unit weight for each batch produced for mixture qualification.

4. Fabricate and cure all specimens in accordance with ASTM C1856, to match the required curing for the structural product.

5. If a thermal treatment will be applied to the product, apply the same thermal treatment to all test specimens, in accordance with ASTM C1856, Section 7.4.

6. An updated Material Identity Card, based on data no older than 2 years, shall be prepared and submitted to the Engineer for approval every 2 years. See Section B.3.1.1.

7. Number of Tests:
   a. A test shall be as defined as in Section B.5.2.7.
   b. Unless otherwise specified, a minimum of nine compressive strength and nine flexural strength tests shall be performed to establish the Material Identity Card. The tests shall represent a minimum of three consecutive batches, with no more than three tests performed for each batch.
   c. Unless otherwise specified, a minimum of one test shall be performed for all other properties and performance characteristics to be reported.

8. Required Average Strength
   a. Determine the required average strength for the following properties as defined below, based on the specified strengths defined in Subsection B.4.3.3 and the sample standard deviations of the test results:
      1. Compressive strength, $f'_{cr}$
      2. First-peak (first crack) flexural strength, $f'_{fcr}$
      3. Peak (ultimate) flexural strength, $f'_{fur}$
   b. Calculate the sample standard deviation, $s$, of the test records.
   c. Calculate the required average strengths based on the test records as follows. Use the larger of the two values calculated.
      $$f'_{X_{r}} = \max \left\{ \frac{f'_{X} + 1.34ks}{0.90f'_{X} + 2.33ks}, \frac{f'_{X} + 2.33ks}{0.90f'_{X} + 2.33ks} \right\}$$
      where $f'_{X}$ is the required average strength for the property $X$; $f'_{X}$ is the specified strength for the property in accordance with Subsection B.4.3.3; $k$ is a modification factor listed in Table B.4.3.5-2 to adjust for the number of tests considered in calculating the sample standard deviation; and $s$ is the sample standard deviation.
   d. The sample standard deviation used to determine the required average compressive strength, first-peak flexural strength, or peak flexural strength shall be determined from a minimum of 9 consecutive tests, with no more than three tests performed from a single batch.

C. Established concrete mix designs for which strength and performance data exist may be used on the basis of past test results if no more than 2 years has passed since testing, the concrete is made from the same material sources, and the same mixing, curing, and thermal treatment procedures will be used for the Project.
Revise “Purpose of Test” in table below to suit Project requirements. “Qualification” tests are required for mixture approval. “Informational - Mandatory” tests are required for submittal but are not required for mixture approval. “Informational - Optional” tests may be performed for informational purposes at the discretion of the producer but are not required for submittal or approval. Revise list to suit Project requirements. Consider permitting testing at 56 days or greater age.

### Table B.4.3.5-1. Qualification and Informational Test Requirements

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Purpose of Test*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength, 28 days</td>
<td>ASTM C1856, with the exception that testing may be performed on 4-inch diameter specimens</td>
<td>Qualification</td>
</tr>
<tr>
<td>Flexural Strength, 28 days</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- First-peak (first crack) (f&lt;sub&gt;fc&lt;/sub&gt;)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Peak (ultimate) (f&lt;sub&gt;fu&lt;/sub&gt;)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Residual at L/300 and L/150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Static Modulus of Elasticity, 28 days</td>
<td>ASTM C1856</td>
<td>Informational - Mandatory</td>
</tr>
<tr>
<td>Poisson's Ratio, 28 days</td>
<td>ASTM C1856</td>
<td>Informational - Optional</td>
</tr>
<tr>
<td>Creep in Compression</td>
<td>ASTM C1856</td>
<td>Informational - Optional</td>
</tr>
<tr>
<td>Length Change</td>
<td>ASTM C1856, with initial reading taken at the time of final set or at 24 hours, whichever occurs last</td>
<td>Informational - Optional</td>
</tr>
<tr>
<td>Time of Set</td>
<td>ASTM C1856</td>
<td>Informational - Optional</td>
</tr>
<tr>
<td>Indication of Resistance to Chloride Ion Penetration, 28 days</td>
<td>ASTM C1856, with tests performed on specimens without metallic fibers</td>
<td>Qualification</td>
</tr>
<tr>
<td>Chloride Diffusion Coefficient, 28 days</td>
<td>ASTM C1556, with tests performed on specimens without metallic fibers</td>
<td>Informational - Optional</td>
</tr>
<tr>
<td>Sulfate Resistance</td>
<td>ASTM C1012, as modified herein</td>
<td>Informational - Optional</td>
</tr>
<tr>
<td>Resistance to Freezing and Thawing</td>
<td>ASTM C1856</td>
<td>Informational - Optional</td>
</tr>
<tr>
<td>Absorption, 28 days</td>
<td>ASTM C642</td>
<td>Informational - Optional</td>
</tr>
</tbody>
</table>

* “Qualification” tests are required for mixture approval. “Informational - Mandatory” tests are required for submittal but are not required for mixture approval. “Informational - Optional” tests may be performed and reported at the discretion of the producer but are not required for submittal or approval.
### B.5. Fabrication

#### B.5.1. Forms

A. **Form**: Accurately construct forms, mortar tight, of sufficient strength to withstand pressures due to concrete placement and vibration operations and temperature changes, and for prestressing and detensioning operations.

1. Design formwork to withstand the full hydrostatic pressure of fresh UHPC.
2. The minimum clear spacing between the formwork surface and any internal reinforcing or adjacent formwork shall be greater than both 1.5 times the fiber length and 1.5 times the maximum aggregate size.

B. **Form-Release Agent**: Coat contact surfaces of forms with release agent before reinforcement is placed. Avoid contamination of reinforcement and prestressing tendons by release agent.

1. Remove excess form release agent prior to placement of the UHPC.

C. Maintain forms to provide completed structural precast concrete members of shapes, lines, and dimensions indicated in Contract Documents, within fabrication tolerances specified.

#### B.5.2. Concrete

A. Do not produce UHPC for construction production until Material Identify Card has been reviewed and approved by Engineer.

##### B.5.2.1. Storage and Handling of Concrete Materials

A. Comply with PCI MNL 116 requirements, with the following modifications:

1. Store and handle all supplementary cementitious materials and mineral fillers in a similar manner to cement.
2. Store and handle aggregates in a wet or dry manner that limits segregation.
3. Store fibers in a dry, covered location to prevent oxidation (steel fibers) or UV degradation (non-metallic fibers). Some surface oxidation (rust) of steel fibers is permissible, provided that the fibers remain as individual strands (i.e., do not clump) and the severity of the oxidation is documented. Discoloration of non-metallic fibers may indicate UV degradation; therefore, discolored non-metallic fibers shall not be used. Elevate fibers stored in bags or sacks on pallets or racks to limit exposure to moisture.
4. Store pre-blended materials in a similar manner to cement. If pre-blending materials on-site, oven-dry aggregates before blending to limit pre-hydration of the cementitious materials.

##### B.5.2.2. Mixing

A. Equipment: Comply with PCI MNL 116 requirements and tolerances for all batching equipment, including scales, water meters, dispensing equipment, and concrete mixers.

---

**Table B.4.3.5-2. k-factor for Increasing Sample Standard Deviation based on Number of Tests**

<table>
<thead>
<tr>
<th>Total number of tests considered</th>
<th>k-factor for increasing sample standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>1.25</td>
</tr>
<tr>
<td>15</td>
<td>1.16</td>
</tr>
<tr>
<td>20</td>
<td>1.08</td>
</tr>
<tr>
<td>25</td>
<td>1.03</td>
</tr>
<tr>
<td>30 or more</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Note: Linear interpolation for intermediate number of tests is acceptable.
B. Mixing: Comply with PCI MNL 116 requirements for batching and mixing, with the following modifications:

1. Mixing time requirements specified in PCI MNL 116 do not apply to UHPC. The time from the start of concrete mixing to placement may exceed 1 hour, as long as the flow requirements are met and the water-binder ratio of the approved mix design is not exceeded.

2. Clean the mixer between consecutive batches of UHPC if the mixer is not loaded within the working time of the previous batch.

3. If aggregates are used in a moist (i.e., not oven-dry) condition, measure aggregate moisture content for each batch. Determine moisture content by calibrated moisture probe, rapid measurement technique (ASTM D4944 or AASHTO T 217), oven-drying (ASTM C566 or AASHTO T 255), or other approved method. Calibrate moisture probes and rapid measurement techniques for each aggregate source.

4. Consider all sources of water in UHPC, including batch water, moisture content in aggregates, and water fraction of admixtures, in determining the Total Water.

5. The batching sequence and mixing procedures shall produce uniform dispersion of the constituent materials and fibers, and achieve the fresh properties required for transporting and placing the UHPC. The batching sequence and mixing procedures do not need to comply with the requirements of PCI MNL 116.

   a. Uniformity of dispersion shall be evaluated, at minimum, by visual inspection of the fresh UHPC at the time of discharge from the mixer. A well-dispersed UHPC mixture will have no fiber clumps and no agglomerates of the powder constituents.

6. Batch materials within tolerances listed in Table B.5.2.2-1.

   **Table B.5.2.2-1. Batching Tolerances**

<table>
<thead>
<tr>
<th>Material</th>
<th>Maximum Batching Error with Plant-Batched Dry Materials</th>
<th>Maximum Batching Error with Pre-Blended Dry Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>± 1%</td>
<td>± 1%</td>
</tr>
<tr>
<td>Cement</td>
<td>± 1%*</td>
<td></td>
</tr>
<tr>
<td>Silica Fume</td>
<td>± 1%*</td>
<td></td>
</tr>
<tr>
<td>Other Supplementary Cementitious Materials or Mineral Fillers</td>
<td>± 1%*, or ± 5 lb, whichever is greater</td>
<td>± 2%</td>
</tr>
<tr>
<td>Aggregates</td>
<td>± 2%*</td>
<td></td>
</tr>
<tr>
<td>Chemical Admixtures</td>
<td>± 2%</td>
<td>± 2%</td>
</tr>
<tr>
<td>Fiber</td>
<td>-2%, +4%</td>
<td>-2%, +4%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
   *Total weight of dry materials shall not exceed ± 2% of target.

C. Mixture Adjustments:

1. Adjustments to water or admixture dosages may be performed to achieve target flow properties. The maximum water-binder ratio in the submitted and approved mixture design shall not be exceeded.

2. No adjustments shall be made after the concrete is discharged from the mixer.

3. Changes to the constituent materials, mixture design, batching equipment, or curing methods will require resubmittal of Material Identify Card to verify compliance with Project requirements. To
limit production delays, provisional approval may be granted at the discretion of the Engineer before 28-day strength results have been obtained.

**B.5.2.3. Transporting, Placing, and Consolidating Concrete**

A. Comply with PCI MNL 116 requirements for transporting and placing concrete, with the following exceptions:

1. The temperature of UHPC at the time of placement shall be between 50 °F (10 °C) and 80 °F (27 °C), unless otherwise approved by the Engineer.
2. Place UHPC in a continuous operation that prevents cold joints or planes of weakness from forming, limits fiber segregation, and reduces the entrapment of air.
3. Use placement methods that limit unfavorable alignment of fibers where possible. Since fibers tend to align in the direction of the UHPC flow, avoid placement methods that cause the UHPC to flow in a direction that is perpendicular to the direction of tensile stress if directed by the Engineer.
4. Place the UHPC in a manner that integrates the new material into the previously placed UHPC. For deep members, use a tremie or other method for ensuring that new material is integrated into previously placed material.
5. Limit free fall placement into the form to a maximum of 3 feet (1 meter) above the placement location.

B. Comply with all PCI MNL 116 requirements for hot and cold-weather concrete placements.

C. Thoroughly consolidate the UHPC without dislocating or damaging the reinforcement (including fibers) and built-in items. Minimize pour lines, honeycombing, and entrapped air voids on the surface. Use consolidation equipment and procedures complying with PCI MNL 116 requirements, with the following exceptions:

1. Do not use internal vibration.
2. Use external vibration only if needed to salvage members when concrete workability is not sufficient to complete placement. If external vibration is used, conduct an investigation per B.5.2.3.D.3 during that placement to determine if detrimental effect on fiber distribution is expected.

D. Evaluate the potential for fiber segregation and settlement during production or as part of a trial placement.

A suggested method for fiber segregation evaluation is provided below. Alternative methods may be specified.

1. Fabricate a specimen mold with a height greater than or equal to the height of the element to be produced, a width equal to that to be used for element, and a length of 1 ft.
2. Place concrete into the specimen mold and consolidate using the same transport, placement, and consolidation methods used for production.
3. After the specimen has hardened, saw-cut the specimen along the centerline over the full height of the specimen. Inspect the cut surface for non-uniform dispersion, including fiber settlement.
4. If non-uniform settlement is identified, establish a lower allowable limit on flow spread for production of elements of the test height or greater.

**B.5.2.4. Finishing**

A. Strike off or screed the surfaces of the UHPC product to the required level immediately after placement. Finish as required by Drawings.

**B.5.2.5. Curing**

A. After finishing, immediately cover all surfaces with plastic, wet burlap, or curing compound to prevent dehydration. Cure the concrete according to PCI MNL 116 requirements, either by moisture retention without heat or by accelerated heat curing using live steam or radiant heat and moisture. Maintain a minimum relative humidity of 95 percent if curing with live steam or radiant heat and moisture.
Implementation of UHPC in Long-Span Precast Pretensioned Elements


1. Accelerated heat curing, if used, shall be started after the concrete has attained initial set, determined in accordance with ASTM C1856.

Cure members until compressive strength is high enough to ensure that stripping does not influence the performance or appearance of the final product.

B. The compressive strength at stripping shall be determined according to Subsection B.4.3.3.

Retain sub-section below if thermal treatment is allowed or required by the Project.

B.5.2.6. Thermal Treatment

A. Apply thermal treatment to members after curing, if used.

Retain the following paragraph. Thermal treatment will cause considerable shrinkage of the product. Thermal treatment performed in the forms may cause cracking.

B. Do not apply thermal treatment until tendons have been detensioned and the UHPC member has been stripped from the forms.

C. Apply thermal treatment according to the regime employed during testing conducted during preparation of Material Identify Card:

1. Heat member at 194 °F (90 °C) and at least 95 percent relative humidity for 48 hours.

2. Thermal treatment may be applied at any time until the member has reached 14 days of age.

3. The rate of heating shall not exceed 36 °F (20 °C) per hour.

4. The rate of cooling after sustained heating shall not exceed 50 °F (27.8 °C) per hour.

B.5.2.7. Inspection and Testing

Always retain paragraph below because it establishes the minimum standard of plant testing and inspecting. PCI MNL 116 mandates source testing requirements and a plant “Quality Systems Manual.” PCI certification also ensures periodic auditing of plants for compliance with requirements in PCI MNL 116.

A. Quality-Control Testing: Test and inspect precast UHPC according to PCI MNL 116 requirements, with the following modifications:

1. A “test” for strength and other hardened properties of the UHPC shall be defined as the average results from at least three specimens made from the same concrete sample and tested at the same age.

2. Perform temperature, flow spread, unit weight, compressive strength, and flexural strength testing according to the procedures defined in Table B.5.2.7-1. Measurement of air content is not required.

3. If test specimens are match-cured, follow procedures of AASHTO R72, except that the specimen sizes, specimen molding, and other testing are to be as defined in this Document.

4. Minimum testing frequencies are listed in Table B.5.2.7-1. Perform additional testing and cast additional samples if a change in quality is suspected.

5. Compressive strength testing shall be performed on 3- or 4-inch diameter specimens.

6. Acceptance of concrete testing shall be as per PCI MNL 116 except the strength of the concrete shall be considered satisfactory provided that both of the following requirements are met:

   a. Every average of three consecutive strength tests equals or exceeds the specified strength, $f'_X$ for the property in accordance with Subsection B.4.3.3.

   b. No single strength test result falls below $f'_X$ by more than 0.10 $f'_X$. 


Implementation of UHPC in Long-Span Precast Pretensioned Elements


Table B.5.2.7-1. Minimum Testing Frequencies

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Minimum Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow Spread</td>
<td>ASTM C1856</td>
<td>Each batch.</td>
</tr>
<tr>
<td>Temperature</td>
<td>ASTM C1064</td>
<td>Each batch.</td>
</tr>
<tr>
<td>Unit Weight</td>
<td>ASTM C138, modified*</td>
<td>First batch per day, whenever test specimens are cast, and whenever a change in quality is suspected.</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>ASTM C1856, with the exception that testing may be performed on 4-inch diameter specimens</td>
<td>6 cylinders from one batch per day per element, or every 25 yd³ (20 m³), whichever is more frequent. Test 3 cylinders at release and 3 cylinders at service.</td>
</tr>
<tr>
<td>First-Peak Flexural Strength ( (f_{fc}) )</td>
<td>ASTM C1856</td>
<td>3 beams from one batch per day per element, or every 25 yd³ (20 m³), whichever is more frequent. Test at service.</td>
</tr>
<tr>
<td>Peak Flexural Strength ( (f_{fu}) )</td>
<td>ASTM C1856</td>
<td></td>
</tr>
<tr>
<td>Residual Flexural Strength</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Fill measure in a single, continuous pour, and consolidate by tapping 30 times with a rubber mallet.

B. Concrete Records: Maintain records of concrete operations consistent with PCI MNL 116 requirements, with the following additions:

1. Maintain reports of each batch of UHPC produced, including quantities of materials weighed, the batching sequence used, the time that concrete was discharged from the mixer, and the time that concrete was placed into the forms.

2. Record all testing performed as specified in Table B.5.2.7-1. Testing and recording of slump and air content are not required for UHPC.

3. Maintain records of any additional testing performed, including any strength testing performed at ages other than at release and at service.

Retain section below if a production mock-up is required for the Project. Revise or add details as needed.

B.5.3. Production Mock-Up

A. To verify that placement and consolidation methods do not adversely affect UHPC uniformity, including fiber distribution and alignment, produce a mock-up element using the placement and consolidation methods intended for the Project.

B. Evaluate uniformity of UHPC fiber at locations specified by the Engineer using one of the following methods.

1. Cut a minimum of three beams at locations specified by the Engineer, with dimensions as required by ASTM C1856, Table 3. Perform flexural strength testing at an age of 28 days, or at the age of service defined by the Engineer, in accordance with ASTM C1856. The tested beams should meet the requirements for flexural strength at service as defined in Subsection B.4.3.3.

2. Cut or core specimens from the concrete at locations specified by the Engineer. Characterize the fiber density and alignment using techniques approved by the Engineer.
C. APPENDIX C - PRECASTER MATERIALS - MILL REPORTS

C.1. Precaster A
Lehigh Cement, a division of Lehigh Hanson Materials Limited  
7777 Ross Road  
Delta, British Columbia, V4G 1B8  
P.O. Box 950, V4K 3S6  
ph: 604.946.0411

**MILL TEST REPORT**

Cement Type: ASTM Type III, AASHTO Type III, CSA Type HE  
High Early Strength, Low Alkali Portland Cement

**Production Period:** Aug 01 2018 to Aug 31 2018

<table>
<thead>
<tr>
<th>Test Result</th>
<th>ASTM C150/C150M-18</th>
<th>AASHTO M 85-16</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SiO₂ (%)</strong></td>
<td>19.9</td>
<td>-</td>
</tr>
<tr>
<td><strong>Al₂O₃ (%)</strong></td>
<td>5.1</td>
<td>-</td>
</tr>
<tr>
<td><strong>Fe₂O₃ (%)</strong></td>
<td>3.32</td>
<td>-</td>
</tr>
<tr>
<td><strong>CaO (%)</strong></td>
<td>63.8</td>
<td>-</td>
</tr>
<tr>
<td><strong>MgO (%)</strong></td>
<td>0.9</td>
<td>max. 6.0</td>
</tr>
<tr>
<td><strong>SO₃ (%)</strong></td>
<td>2.82</td>
<td>max. 3.5</td>
</tr>
<tr>
<td><strong>Na₂O (%)</strong></td>
<td>0.27</td>
<td>-</td>
</tr>
<tr>
<td><strong>K₂O (%)</strong></td>
<td>0.38</td>
<td>-</td>
</tr>
<tr>
<td><strong>TiO₂ (%)</strong></td>
<td>0.29</td>
<td>-</td>
</tr>
</tbody>
</table>

| **C₃S (%)**  | 59                | -              |
| **C₂S (%)**  | 11                | -              |
| **C₃A (%)**  | 8                 | max. 15        | -        |
| **C₄AF (%)** | 0.51              | max. 0.60      | -        |

| **Loss on Ignition (%)**| ASTM C314 | ASTM C315 | CSA 45003 | CSA 45003 | 2.5 | max. 3.5 | max. 3.5 |
| **Insoluble Residue (%)** | ASTM C314 | ASTM C315 | CSA 45003 | CSA 45003 | 0.14 | max. 1.5 | max. 1.5 |
| **Free Calcium Oxide (%)** | ASTM C314 | ASTM C315 | CSA 45003 | CSA 45003 | 0.5 | - | - |
| **CO₂ in Cement (%)** | ASTM C314 | ASTM C315 | CSA 45003 | CSA 45003 | 1.4 | - | - |
| **CaCO₃ in Limestone (%)** | ASTM C314 | ASTM C315 | CSA 45003 | CSA 45003 | 97 | min. 70 | - |
| **Limestone in Cement (%)** | ASTM C315 | ASTM C315 | CSA 45003 | CSA 45003 | 3.4 | max. 5.0 | - |

| **Vicat Setting Time (min)** | ASTM C191 | ASTM C304-B2 | CSA 4504-B2 | CSA 4504-B2 | 89 | min. 45, max. 157 | min. 45, max. 250 |
| **Blaine Fineness (m²/kg)** | ASTM C204 | ASTM C204 | CSA 4504-A1 | CSA 4504-A1 | 533 | - | - |
| **+325 mesh** | ASTM C401 | ASTM C401 | CSA 4504-A1 | CSA 4504-A1 | 3.1 | - | - |
| **Air Content of Mortar (%)** | ASTM C305 | ASTM C305 | CSA 4504-C4 | CSA 4504-C4 | 5.2 | max. 12 | - |
| **Autoclave Expansion (%)** | ASTM C311 | ASTM C311 | CSA 4504-B3 | CSA 4504-B3 | 0.60 | max. 0.80 | max. 1.0 |

| **Compressive Strength (MPa/psi)** | ASTM C300/1050M | ASTM C3004-C2 | CSA 4504-C2 | CSA 4504-C2 | 21.1 / 3060 | min. 12.0 | min. 13.5 |
| **1 Day** | ASTM C300/1050M | ASTM C3004-C2 | CSA 4504-C2 | CSA 4504-C2 | 38.1 / 5530 | min. 24.0 | min. 24.0 |
| **28 Day (previous month)** | ASTM C300/1050M | ASTM C3004-C2 | CSA 4504-C2 | CSA 4504-C2 | 53.4 / 7740 | - | - |

This will certify that the above described cement meets the standard requirements and optional Low Alkali chemical requirements of ASTM Specification C-150 and AASHTO Specification M-85 for Type III Portland Cement, and CSA Specification A2001 and A5 for Type HE Portland Cement.


Our Laboratory is AASHTO-accredited.

Siu Kei (S.K.) Ng  
Plant Chemist  
September 12, 2018

Appendix C – Precaster Materials – Mill Reports
FORCE 10,000® D

High performance concrete admixture dry densified powder

Product Description

FORCE 10,000® D is a dry densified microsilica (silica fume) powder designed to increase concrete compressive and flexural strengths, increase durability, reduce permeability and improve hydraulic abrasion-erosion resistance. The specific gravity of FORCE 10,000® D is 2.20.

Uses

FORCE 10,000® D can be used to consistently produce concrete with strengths of 6,000 psi (42 MPa) and higher in most instances with locally available materials and existing methods. It may also be used in precast and prestress applications where high early strengths are required.

The addition of FORCE 10,000® D also produces concrete with increased watertightness and dramatically reduced permeability compared to conventional mixes. Reduced permeability is an important advantage in slowing the intrusion of chloride where corrosion of reinforcing steel is a potential problem. Examples are parking garages, bridge decks and concrete in a marine environment. FORCE 10,000® D also enhances the durability of concrete against aggressive chemical attack and in hydraulic abrasion-erosion applications.

Preconstruction Trial Mix

It is strongly recommended that trial mixes be made several weeks before construction start up. This will allow the concrete producer an opportunity to determine the proper batching sequence and amounts of other admixtures needed in order to deliver the required concrete mix to the job site. A trial mix will also help determine whether the combination of concrete materials and construction practices will allow the concrete to meet a specified performance. GCP's broad experience with this product can help the concrete producer deliver a satisfactory product regardless of the mixture proportions. Contact your GCP Applied Technologies sales representative for help with trial mixes.

Finishing & Curing

FORCE 10,000® D concrete can be used in flatwork with little or no modification to the recommended practices outlined in ACI 302, Guide for Concrete Floor and Slab Construction.

FORCE 10,000® D will reduce the surface bleed water of concrete in large applications. ACI 308, Standard Practice for Curing Concrete, must be followed to ensure that any problems that can occur due to decreased bleeding are minimized. Your GCP Applied Technologies representative is available to review your particular job needs.
Performance

FORCE 10,000® D improves concrete through two mechanisms. The extremely fine microsilica particles are able to fill the microscopic voids between the cement particles, creating a less permeable structure. In addition, the microsilica reacts with the free calcium hydroxide within the concrete to form additional calcium silicate hydrate (glue), producing a tighter paste-to-aggregate bond. FORCE 10,000® D does not affect concrete set times.

FORCE 10,000® D will improve the mechanical properties of concrete. In order to meet specified concrete performance levels, however, many variables are involved. These include, but are not limited to, concrete materials, weather conditions, testing techniques and mixing, transporting, placing and finishing practices. ACI and ASTM guidelines must be strictly adhered to.

Addition Rates

FORCE 10,000® D dosage rates will vary based on the requirements of the application. Dosage rates should be calculated on percent microsilica by weight of cement, or on lb/yd$^3$ (kg/m$^3$) of concrete, as appropriate. Dosage rates will be as specified. If not specified, consult your GCP Applied Technologies representative for your particular job needs.

Compatibility with Other Admixtures and Batch Sequencing

FORCE 10,000® D is compatible with all conventional water reducers, superplasticizers, set retarders and DCI® corrosion inhibitor. Any air-entraining agent which works effectively with superplasticizers and microsilica, particularly vinyl-sol resins such as DARAVAIR® by GCP Applied Technologies, are recommended. Only non-chloride set accelerators, such as POLARSET®, may be used with FORCE 10,000® D concrete. All admixtures must be added separately to assure their prescribed performance. Trial mixes and pretesting of concrete are recommended to optimize dosage rates, and ensure ultimate performance.

FORCE 10,000® D can be used in either central or transit mix concrete production. FORCE 10,000® D may be used in conjunction with water-reducing admixtures (both normal and high-range as approved by ASTM) to assure workability of the mix.

Packaging, Handling and Storage

FORCE 10,000® D is available in bulk, and 25 lbs (11.4 kg) Concrete Ready Bags™.

Bagged FORCE 10,000® D should be stored in a dry, protected area. Manual dispensing by tearing the bags is the normal method. A dust mask should be used when dispensing the bagged product, consult the product MSDS for more complete instructions.
Dispensing Equipment

Bulk FORCE 10,000® D may be stored in already existing cement silos. The silos must be completely clean with no foreign residue remaining which may cause contamination. Up-pipes to the silo for unloading bulk tankers should also be clean and clear of obstructions. Small diameter 4 in. (100 mm) rigid metal pipes with several angles (especially right angles) will cause longer unloading times. Large diameter 6 in. (150 mm) flat lined, flexible rubber pipes will allow for the least unloading time. Dispensing bulk FORCE 10,000® D will take place in the same manner as that used for cement. Augering or dropping from the silo to the weigh hopper is the usual practice.
### Appendix C – Precaster Materials – Mill Reports

#### SEPTEMBER 2018 AVERAGES

##### GRAVEL

<table>
<thead>
<tr>
<th>PRODUCT #</th>
<th>MATERIAL</th>
<th>U.S. 2&quot;</th>
<th>1 ½&quot;</th>
<th>1&quot;</th>
<th>3/4&quot;</th>
<th>5/8&quot;</th>
<th>1/2&quot;</th>
<th>3/8&quot;</th>
<th># 4</th>
<th># 10</th>
<th># 200</th>
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<tr>
<td># 8428</td>
<td>1 ½&quot; x 1&quot;</td>
<td>50.0</td>
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<td># 8490</td>
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<td># 8493</td>
<td>AASHTO #8</td>
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<td># 8495</td>
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##### SAND

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<th># 8</th>
<th># 16</th>
<th># 30</th>
<th># 100</th>
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##### CSBC & CSTC CRUSHED GRAVEL

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<th># 10</th>
<th># 40</th>
<th># 200</th>
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<td>44</td>
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<td># 8821</td>
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<td>87</td>
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<td>52</td>
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##### CRUSHED GRAVEL

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<td>0.5</td>
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<td># 8870</td>
<td>4 to Dust</td>
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<td>89</td>
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<td>10.8</td>
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##### FILL

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<tr>
<th>PRODUCT #</th>
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<th>3/4&quot;</th>
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<th>1/2&quot;</th>
<th># 4</th>
<th># 10</th>
<th># 40</th>
<th># 200</th>
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<tbody>
<tr>
<td># 8177</td>
<td>Seattle Type 17</td>
<td>100</td>
<td>96</td>
<td>97</td>
<td>87</td>
<td>65</td>
<td>44</td>
<td>30</td>
<td>17</td>
<td>7.2</td>
</tr>
<tr>
<td># 8128</td>
<td>Gravel Borrow</td>
<td>100</td>
<td>96</td>
<td>97</td>
<td>93</td>
<td>74</td>
<td>63</td>
<td>31</td>
<td>11</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Please contact us if you need a special blend or the range of a specific product. The results above are an average & individual tests may vary.

Phone: (255) 912-8500
Fax: (253) 912-8510

DuPont Pit # B-335

#8 = 1% Passing
#493 AASHTO #8
#8 = 2% Passing
#16 = 3% Passing

eConstruct-WJE-UNL-NCSU 271 January 20
HRWR – High Cementitious Content Concrete

**Features**

**CHRYSO® Fluid Premia 150** is a new generation high range water reducing admixture based on modified polycarboxylates.

**CHRYSO® Fluid Premia 150** is formulated specifically to allow for manufacturing of very high cementitious content concrete and achieve superior strength performance at all ages.

**CHRYSO® Fluid Premia 150** exclusive formulation allows for extreme easiness of use and robustness.

**CHRYSO® Fluid Premia 150** is manufactured under rigid quality control standards to provide uniform, reliable results.

**Benefits**

- Provides enhanced workability retention
- Provides increased slump and flowability without increased water content
- Improves finish and placement of concrete
- Allows for high strength performance at all ages
- Improves concrete quality by reducing the water-cement ratio for a given degree of workability
- Proprietary molecule allows for easiness of use and concrete performance consistency
- Reduces cracking and shrinkage
- Improves concrete chemical resistance and durability
- Improves cementitious material performance (more psi/lb)

**Areas of Application**

**CHRYSO® Fluid Premia 150** is recommended for all concrete mixes where significant water reduction, improved cementitious material performance (more psi/lb), improved finishing, good slump retention and enhanced flowability characteristics are desirable including SCC.

**CHRYSO® Fluid Premia 150** is especially recommended for use in Ultra High Performance Concrete applications where high flowability characteristics, high strengths at all ages and extended workability are required.
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix C – Precaster Materials – Mill Reports

CHRYSO® Fluid Premia 150

Description:

- **Compatible with all types of Portland cement, class C and F fly ash, slag, microsilica, calcium chloride, fibers and approved air entraining admixtures.**

- **May freeze at temperatures below 28°F (-2°C). Although freezing does not harm CHRYSO® Fluid Premia 150, precautions should be taken to protect it from freezing.**

- **If CHRYSO® Fluid Premia 150 should happen to freeze, thaw and reconstitute with mechanical agitation.**

- **Do not store the product at temperatures above 100°F (38°C) or under 40°F (5°C) for long periods.**

- **Shelf life: 9 months.**

Directions for use:

- **CHRYSO® Fluid Premia 150 is recommended for use at a dosage rate of 3 to 6 fluid ounces per 100 pounds (185 to 361 ml per 100 kg) of cementitious material for a Type A and 5 to 40 fluid ounces per 100 pounds (326 to 2600 ml per 100 kg) of cementitious material for a Type F.**

- **CHRYSO® Fluid Premia 150 can be added at the concrete plant with the initial or tail water or on the job site. In case of addition in a mixing truck, it is recommended that the concrete be mixed at high speed for approximately 3 – 5 minutes.**

- Because local job conditions vary, please contact your local CHRYSO sales representative for further assistance if using outside recommended dosage ranges.

CHRYSO INC
Tel: (860) 936-7553 – 972-772-6010
Southern Division P.O. Box 190 Rockwall, TX 75032
Midwest Division P.O. Box 129 Charlestown, IN 47111
Western Division 5890 Rome St Denver, CO 80239

This information contained in this document is given in good faith and is the result of extensive and controlled testing. However, it cannot and/or any determinations be considered a warranty implying our liability in the event of failure. Tests should be conducted before the product is used to ensure that the methods and conditions of use of the product are satisfactory. Our specialists remain at the disposal of customers if they require help with the application of the product for their specific needs.

About CHRYSO:

A worldwide leader for Concrete and Cement additives, CHRYSO has been servicing the construction industry for over half a century with outstanding innovation and service. As a result, CHRYSO’s name and products have been associated with the most prestigious and demanding construction projects worldwide.

www.chrysoinc.com
C.2. Precaster B
Cement Mill Test Report  
Month of Issue: October 2018

Plant: Harleyville, South Carolina  
Product: Portland Cement Type I and Type III(MH)  
Silo: 1, 4, 5, 7, 9  
 Manufactured: September 2018

### ASTM C150 and AASHTO M 85 Standard Requirements

#### CHEMICAL ANALYSIS

<table>
<thead>
<tr>
<th>Item</th>
<th>Spec limit</th>
<th>Test Result</th>
</tr>
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<tbody>
<tr>
<td>Rapid Method, X-Ray (C714)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SiO₂ (%)</td>
<td>---</td>
<td>20.0</td>
</tr>
<tr>
<td>Al₂O₃ (%)</td>
<td>---</td>
<td>6.0 max</td>
</tr>
<tr>
<td>Fe₂O₃ (%)</td>
<td>---</td>
<td>4.9</td>
</tr>
<tr>
<td>CaO (%)</td>
<td>---</td>
<td>6.0 max</td>
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<tr>
<td>MgO (%)</td>
<td>---</td>
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<td>SO₃ (%)</td>
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<tr>
<td>Loss on ignition (%)</td>
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<td>3.0 max</td>
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<tr>
<td>Insoluble residue (%)</td>
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<tr>
<td>CO₂ (%)</td>
<td>---</td>
<td>1.0 max</td>
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<tr>
<td>Limestone (%)</td>
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<tr>
<td>Ga₂O₃ in Limestone (%)</td>
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<td>Inorganic Processing Addition (%)</td>
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#### PHYSICAL ANALYSIS

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<td>Air content of mortar (%) (C165)</td>
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<td>12 max</td>
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<td>Blaine Fineness (m²/kg) (C204)</td>
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<td>200 - 400</td>
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<td>-325 (%) (C-430)</td>
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<td>12 days (max)</td>
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<tr>
<td>Autoclave expansion (%) (C155)</td>
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<tr>
<td>Compressive strength (MPa, [PSI]) (C709)</td>
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<td>26.0</td>
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<tr>
<td>3 days</td>
<td>---</td>
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<td>7 days</td>
<td>---</td>
<td>10.0</td>
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<td>28 days</td>
<td>---</td>
<td>12.0</td>
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<tr>
<td>28 days (Reflects previous month's data)</td>
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<td>14.0</td>
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<td>Time of setting (minutes)</td>
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<td>7 days</td>
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<tr>
<td>Mortar Bar Expansion (%) (C1938)</td>
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<td>Density (g/m³)</td>
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#### ASTIM C150 and AASHTO M 85 Optional Chemical Requirements:

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<td>Na₂O (%)</td>
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<td>Chloride (%)</td>
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* Dashes in the spec limit mean Not Applicable
1 May exceed 3.0% in order based on our C1034 results of < 0.021% expansion at 14 days.
2 Current production run not available - most recent provided.

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirements of applicable GDOT, TDOT, MDOT, SCDOT, NCDOT, and VDOT Specifications for Type I and Type III(MH) and low alkali cement.

ASTM C150 & AASHTO M 85 STANDARD SPECIFICATIONS FOR TYPE I AND TYPE III(MH) CEMENT:

ASTM C150 & AASHTO M 85 OPTIONAL CHEMICAL REQUIREMENTS FOR TYPES I & III(MH) LOW ALKALI CEMENT:

Certified By:

[Signature]

Sean J. Makens - Quality Manager
Report created: 10/15/2018
# Cement Mill Test Report

**Month of Issue: October 2018**

**Plant:** Harleyville, South Carolina  
**Product:** Portland Cement Type III  
**Silo:** 6, 8  
**Manufactured:** September 2018

## ASTM C150 and AASHTO M 85 Standard Requirements

### CHEMICAL ANALYSIS

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<td>Al₂O₃ (%)</td>
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<tr>
<td>Fe₂O₃ (%)</td>
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<td>63.6</td>
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<td>CaO (%)</td>
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<td>MgO (%)</td>
<td>3.5 max</td>
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<td>Loss on Ignition (%)</td>
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<tr>
<td>Insoluble residue (%)</td>
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### PHYSICAL ANALYSIS

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<th>Spec limit</th>
<th>Test Result</th>
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<tbody>
<tr>
<td>Air content of mortar (%) (C185)</td>
<td>12 max</td>
<td>8</td>
</tr>
<tr>
<td>Blaine Fineness (m2/kg) (C204)</td>
<td>---</td>
<td>545</td>
</tr>
<tr>
<td>-325 (%) (C430)</td>
<td>---</td>
<td>99.8</td>
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<tr>
<td>Autoclave expansion (%) (C155)</td>
<td>0.80 max</td>
<td>-0.03</td>
</tr>
<tr>
<td>Compressive strength (MPa, [PSE]) (C199)</td>
<td>12.0 [1740] min</td>
<td>23.4 [3400]</td>
</tr>
<tr>
<td>Time of setting (minutes)</td>
<td>Vical Initial (C191)</td>
<td>45 - 375</td>
</tr>
<tr>
<td>28 days (Reflects previous month’s data)</td>
<td>47.0 [6090]</td>
<td></td>
</tr>
</tbody>
</table>

**Adjusted Potential Phase Composition (C150):**

- C₃S (%) --- 56
- C₂S (%) --- 16
- C₃A (%) 15 max 8
- C₄AF (%) --- 10

**ASTM C150 and AASHTO M 85 Optional Chemical Requirements:**

- Na₂O (%) 0.60 max 0.51
- Chloride (%) 0.03

* Dashes in the spec limit mean Not Applicable
* May exceed 3.5% SO₃ maximum based on our C1098 results of < 0.326% expansion at 14 days.
* Current production ran not available - most recent provided.

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirements of applicable SCDOT, NCDOT, CODOT, MDOT, VDOT, DOT, TOOT, FDOT Specifications for Type III CEMENT:

**ASTM C150 & AASHTO M 85 STANDARD SPECIFICATIONS FOR TYPE III CEMENT:**

**ASTM C150 & AASHTO M 85 OPTIONAL CHEMICAL REQUIREMENTS FOR TYPES III LOW ALKALI CEMENT.**

Certified By:

Argos USA  
Harleyville Plant  
463 Judge Street, Harleyville, South Carolina 29448  
Phone: 843-462-7651

Sean J. Makens - Quality Manager  
Report created: 10/15/2018
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix C – Precaster Materials – Mill Reports

BASF
We create chemistry

MasterLife® SF100
Silica fume mineral admixture

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>FEATURES AND BENEFITS</th>
</tr>
</thead>
</table>
| MasterLife SF100 is a dry, compacted, silica fume mineral admixture formulated to produce concrete with special performance qualities. It improves the hardened characteristics of concrete in two main ways. Firstly, MasterLife SF100 is a pozzolan which reacts chemically to increase the amount of calcium silicatehydrate gel formed, thus improving the strength and impermeability of the concrete. Secondly, MasterLife SF100 is an ultra-fine material that physically fills the voids between cement particles resulting in an extremely dense and impermeable concrete. MasterLife SF100 meets AS/NZS 3592.3 Amorphous Silica and ASTM C 1240 requirements. | MasterLife SF100 aids in the production of concrete with the following special qualities:  
- Dramatically improved durability  
- Uniformly high compressive strength  
- Abrasion and erosion protection  
- Better flexural strengths at all ages  
- Low permeability  
- Excellent freeze/thaw resistance |

<table>
<thead>
<tr>
<th>RECOMMENDED USES</th>
<th>TYPICAL PERFORMANCE DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>The reduced permeability of concrete produced with MasterLife SF100 greatly limits the ingress of water, chlorides, sulphates, and aggressive chemicals known for promoting reinforcing steel corrosion and other distress in the concrete. This makes MasterLife SF100 an ideal product for use in basement structures, parking decks, bridge decks, marine structures, and any construction that requires the protection provided by impermeable concrete. It allows for design flexibility, resulting in reduced member size, increased span lengths, and improves overall structural economics which strengthen concrete. As a result of the preceding advantages, MasterLife SF100 will improve performance in wet and dry shotcrete, prestressed, precast, and ready-mixed concrete applications. When air entrainment is desired, an air-entraining admixture is recommended. Please consult your local BASF Technical Sales Representative. It is also recommended that MasterLife SF100 be used in conjunction with a high range, water-reducing admixture, such as Master Rheobuild or MasterGlenium admixtures for maximum workability while maintaining a low water-cementitious ratio.</td>
<td>Example of the influence of MasterLife SF100 on rapid chloride permeability.</td>
</tr>
</tbody>
</table>

![Chloride Permeability Table](image)

**COMPATABILITY**

MasterLife SF100 silica fume can be used with Portland cements approved under AS, NZS, AS/NZS and similar international standards and specifications. It is compatible with most concrete admixtures, including all BASF Construction Chemicals admixtures. MasterLife SF100 silica fume is recommended for use in combination with high-range water-reducers, such as Rheobuild and Glenium admixtures, for maximum workability while maintaining a low water-cementitious materials ratio.
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix C – Precaster Materials – Mill Reports

MasterLife® SF100

RATE OF HARDENING
Setting time of concrete is influenced by the chemical and physical composition of the cement and/or cement type used to produce the concrete, temperature of the concrete, weather conditions, and the use of chemical admixtures. Trial mixes should be made with the job materials to determine the setting time of a specific mixture. MasterLife SF100 silica fume will not initiate or promote corrosion of reinforcing steel embedded in concrete, prestressed concrete or concrete placed on galvanized steel floor and roof systems. Neither calcium chloride nor any chloride-based ingredients are used in the manufacture of MasterLife SF100 silica fume.

DISPENSING
MasterLife SF100 is batched at the ready-mix plant in a manner similar to cement or other cementitious materials such as fly ash and granulated slag. MasterLife SF100 is supplied in biodegradable bags and can be placed directly into the agitator. It is recommended that MasterLife SF100 be used with a BASF high-range water reducer in order to provide maximum workability while maintaining the desired low water/cement ratio.

Note: For directions on the proper use of MasterLife SF100 specific applications, contact your local BASF Technical Sales Representative.

DOSAGE
MasterLife SF100 is recommended for use at an addition rate of 5 to 15% by weight of cement, depending on the amount of strength increase or impermeability desired. The exact amount for strength or durability requirements should be determined by trial batches using project materials.

PACKAGING
MasterLife SF100 is available in 20kg degradable bag.

STORAGE/ SHELF LIFE
MasterLife SF100 can be kept for 12 months if stored in original packaging, in a cool dry place and protected against physical damage.

PRECAUTIONS
For the full health and safety hazard information and how to safely handle and use this product, please make sure that you obtain a copy of the BASF Material Safety Data Sheet (MSDS) from our office or our website.

STATEMENT OF RESPONSIBILITY
The technical information and application advice given in this BASF publication are based on the present state of our best scientific and practical knowledge. As the information herein is of a general nature, no assumption can be made as to a product's suitability for a particular use or application and no warranty as to its accuracy, reliability or completeness either expressed or implied is given other than those required by law. The user is responsible for checking the suitability of products for their intended use.

NOTE
Field service where provided does not constitute supervisory responsibility. Suggestions made by BASF either orally or in writing may be followed, modified or rejected by the owner, engineer or contractor since they, and not BASF, are responsible for carrying out procedures appropriate to a specific application.

BASF Construction Chemicals offices in ASEAN

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<tr>
<th>Country</th>
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<th>Phone 2</th>
<th>Phone 3</th>
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<tr>
<td>Singapore</td>
<td>+65-6881-6766</td>
<td>+65-6881-3186</td>
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<tr>
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### Typical Product Data

**Region:** MID ATLANTIC DIVISION  
South Carolina District

**Quarry:** Calhoun Sand  
110 Access Rd  
Gaston, SC 29053

**Rock Type:** Sand

---

#### Fine Aggregate

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<tr>
<th>Material</th>
<th>Loose Mass</th>
<th>Bulk SSD</th>
<th>Bulk Dry</th>
<th>Soundness Loss</th>
<th>Absorption</th>
<th>Sand Equivalent</th>
<th>Particle Shape Index (Method A)</th>
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<tbody>
<tr>
<td>Fill Sand</td>
<td>85.4</td>
<td>2.552</td>
<td>2.593</td>
<td>13.1%</td>
<td>1.69%</td>
<td>99</td>
<td>42.8%</td>
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</table>

<table>
<thead>
<tr>
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<th>Bulk Dry</th>
<th>Soundness Loss</th>
<th>Absorption</th>
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<tr>
<td>Mason Sand</td>
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<td>2.584</td>
<td>2.594</td>
<td>19.8%</td>
<td>28</td>
<td>98</td>
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<table>
<thead>
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<th>Bulk Dry</th>
<th>Soundness Loss</th>
<th>Absorption</th>
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<th>Particle Shape Index (Method A)</th>
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<tbody>
<tr>
<td>Concrete Sand</td>
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<table>
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<th>Bulk Dry</th>
<th>Soundness Loss</th>
<th>Absorption</th>
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<th>Particle Shape Index (Method A)</th>
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</thead>
<tbody>
<tr>
<td>Sand Clay</td>
<td>72.3</td>
<td>2.362</td>
<td>2.395</td>
<td>22.6%</td>
<td>7.23%</td>
<td>95</td>
<td>47.2%</td>
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</tbody>
</table>

---

ASTM Methods: C29 (Unit Weight), C479 (Aggregate), F136 (Sand), F1361 (Aggregate), F428 (Fine Aggregate), F962 (Particle Shape Index), D692 (Sand Equivalent)

---

*Kyle T. Brashares*  
Asst. Lab Manager  
MLE/C (CTY)
MasterGlenium® 7920
High-Range Water-Reducing Admixture

Description
MasterGlenium 7920 ready-to-use high-range water-reducing admixture is based on the next generation of polycarboxylate technology. This technology incorporates state-of-the-art molecular engineering to provide fast wet-out of powder materials.

MasterGlenium 7920 admixture is effective in improving the day-to-day production efficiency of a concrete plant by rapidly dispersing powder materials in concrete mixtures, thereby minimizing mixing time. It is formulated to meet ASTM C 494 requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures.

Applications
Recommended for use in:
- Concrete requiring high-early compressive strength development
- Concrete mixtures with low w/cm and/or high powder contents
- Production of self-consolidating concrete (SCC) mixtures
- Green Sense® Concrete

Features
- Dosage flexibility
- Rapid cement dispersion
- Superior early and ultimate strengths

Benefits
- Optimized mixture costs
- Shorter mixing time in central mixers
- Increased productivity
- Greater batch-to-batch consistency

Guidelines for Use
Dosing: MasterGlenium 7920 admixture has a recommended dosage range of 2-12 fl oz/cwt (130-760 mL/100 kg) of cementitious materials. For most applications, dosages in the range of 2-8 fl oz/cwt (130-520 mL/100 kg) will provide excellent performance. For very high performance SCC mixtures, up to 12 fl oz/cwt (780 mL/100 kg) of cementitious materials can be utilized. Because of variations in concrete materials, job site conditions and/or applications, dosages outside of this range may be required. In such cases, contact your local sales representative.

Mixing: MasterGlenium 7920 admixture can be added with the initial batch water or as a delayed addition. However, optimum water reduction is generally obtained with a delayed addition.

Product Notes
Corrosivity – Non-Chloride, Non-Corrosive: MasterGlenium 7920 admixture will neither initiate nor promote corrosion of reinforcing steel embedded in concrete, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of MasterGlenium 7920 admixture.

Compatibility: MasterGlenium 7920 admixture is compatible with most admixtures used in the production of quality concrete, including normal, mid- and high-range water reducers, accelerators, retarders, extended-set control admixtures, air entrainers, corrosion inhibitors, and shrinkage reducers. Do not use MasterGlenium 7920 admixture with admixtures containing beta-naphthalene sulfonate. Erratic behaviors in slump, workability retention and pumpability may be experienced.
MasterGlenium 7920

Technical Data Sheet

Storage and Handling

Storage Temperature: MasterGlenium 7920 admixture must be stored at temperatures above 40 °F (5 °C). If MasterGlenium 7920 admixture freezes, thaw and reconstitute by mechanical agitation. Do not use pressurized air for agitation.

Shelf Life: MasterGlenium 7920 admixture has a minimum shelf life of 6 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of MasterGlenium 7920 admixture has been exceeded.

Packaging

MasterGlenium 7920 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Safety Data Sheets: MasterGlenium 7920 admixture

Additional Information

For additional information on MasterGlenium 7920 admixture or on its use in developing concrete mixtures with special performance characteristics, contact your BASF representative.

The Admixture Systems business of BASF’s Construction Chemicals division is the leading provider of solutions that improve placement, pumping, finishing, appearance and performance characteristics of specialty concrete used in the ready-mixed, precast, manufactured concrete products, underground construction and paving markets. For over 100 years we have offered reliable products and innovative technologies, and through the Master Builders Solutions brand, we are connected globally with experts from many fields to provide sustainable solutions for the construction industry.

Limited Warranty Notice

BASF warrants this product to be free from manufacturing defects and to meet the technical properties on the current Technical Data Guide, if used as directed within shelf life. Satisfactory results depend not only on quality products but also upon many factors beyond our control. BASF MAKES NO OTHER WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE WITH RESPECT TO ITS PRODUCTS. The sole and exclusive remedy of Purchaser for any claim concerning this product, including but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is shipment to purchaser of product equal to the amount of product that fails to meet this warranty, at the sole option of BASF. Any claims concerning this product must be received in writing within one (1) year from the date of shipment and any claims not presented within that period are waived by Purchaser. BASF WILL NOT BE RESPONSIBLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDING LOST PROFITS) OR PUNITIVE DAMAGES OF ANY KIND.

Purchaser must determine the suitability of the products for the intended use and assumes all risks and liabilities in connection therewith. This information and all further technical advice are based on BASF's present knowledge and experience. However, BASF assumes no liability for providing such information and advice including the extent to which such information and advice may relate to existing third party intellectual property rights, especially patent rights, nor shall any legal relationship be created by or arise from the provision of such information and advice. BASF reserves the right to make any changes according to technological progress or further developments. The Purchaser of the Product(s) must test the product(s) for suitability for the intended application and purpose before proceeding with a full application of the product(s). Performance of the product described herein should be verified by testing and carried out by qualified experts.
C.3. Precaster C
## Implementation of UHPC in Long-Span Precast Pretensioned Elements

### Appendix C – Precaster Materials – Mill Reports

**CERTIFICATE OF QUALITY**

**PRODUCT**

PORTLAND CEMENT

**TYPE**

I-II (MH) LOW ALKALI

**MANUFACTURING FACILITY**

PATRAS PLANT [Titan Group]

**SILO(S)**

MV JIA HUI SHAN

### STANDARD REQUIREMENTS

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<tr>
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<tr>
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<tr>
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<td>-</td>
</tr>
<tr>
<td>C4AF (%)</td>
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<tr>
<td>C4AF + 0.6(C3A) (%)</td>
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<td>-</td>
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<tr>
<td>C3S (%)</td>
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### PHYSICAL

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<th>Item</th>
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<th>Results for Period</th>
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<tbody>
<tr>
<td>% Air content of mortar</td>
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<td>12</td>
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<tr>
<td>Free water (Mg/kg)</td>
<td>max 260</td>
<td>260</td>
</tr>
<tr>
<td>Blaine max - 300</td>
<td>max 412</td>
<td>412</td>
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<tr>
<td>% Autoclave expansion</td>
<td>max 0.80</td>
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<tr>
<td>Compressive strength (psig)</td>
<td>1 day</td>
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</tr>
<tr>
<td></td>
<td>3 days</td>
<td>2410</td>
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<td></td>
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### OPTIONAL REQUIREMENTS

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<td>KOH (%)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Al2O3 (%)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>LOI (%)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CO2 (%)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

### NOTES:

A. This product conforms to the referenced standard(s) as Type I, Type II, all Low Alkali.
B. This product conforms to Section 921 of the Florida Department of Transportation Standard Specifications.
C. The data in the certificate reflects the results of compliance testing. The data is provided for information only.
D. This product does not contain an inorganic process addition.
E. This product may contain up to 5% limestone addition and the actual test result may vary on spec samples.
F. Test result represents most recent value and is provided for information only.
G. This product and the concrete used in its production were manufactured in Greece.

---

Cynthia Jimenez  
QC Manager

Form Rev. 11 May 2018
CERTIFICATE OF QUALITY

PRODUCT: PORTLAND CEMENT
TYPE: I-II (MH) LOW ALKALI

MANUFACTURING FACILITY: PATRAS PLANT (Titan Group)
SILO(S): MV Same Lion

DATE: December 3, 2018
PERIOD: 10/03/18 to 10/11/18 (Date of Loading)

ADDITIONAL DATA

<table>
<thead>
<tr>
<th>Type</th>
<th>Amount (%)</th>
<th>Limestone</th>
<th>Inorganic Processing Addition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amount (%)</td>
<td>4.3</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>SiO₂ (%)</td>
<td>6.6</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>Al₂O₃ (%)</td>
<td>1.3</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>Fe₂O₃ (%)</td>
<td>1</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>CaO (%)</td>
<td>49.8</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>SO₃ (%)</td>
<td>&lt;0.1</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td>CO₂ (%)</td>
<td>40</td>
<td>---</td>
<td></td>
</tr>
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Base cement Phase Compositions

<table>
<thead>
<tr>
<th>Type</th>
<th>Amount (%)</th>
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</thead>
<tbody>
<tr>
<td>C3S</td>
<td>61</td>
</tr>
<tr>
<td>C2S</td>
<td>13</td>
</tr>
<tr>
<td>C3A</td>
<td>5</td>
</tr>
<tr>
<td>C4AF</td>
<td>13</td>
</tr>
</tbody>
</table>

We certify that the above described data represents the materials used in the cement shipped on the period indicated.

Cynthia Jimenez
QC Manager

Form Rev. 03 FEB 2011
### SILICA FUME CHEMICAL & PHYSICAL ANALYSIS REPORT

All Testing per ASTM C-1240

<table>
<thead>
<tr>
<th>CUSTOMER:</th>
<th>BASF Construction Chemicals LLC</th>
</tr>
</thead>
<tbody>
<tr>
<td>DESTINATION:</td>
<td>Precaster C</td>
</tr>
<tr>
<td>DATE:</td>
<td>5/8/16</td>
</tr>
<tr>
<td>QUANTITY:</td>
<td>32,000 lbs.</td>
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<tr>
<td>SHIPPING #:</td>
<td>60680</td>
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<tr>
<td>N. 1. #:</td>
<td>60294</td>
</tr>
<tr>
<td>LOT #:</td>
<td>14V23D05-2A</td>
</tr>
<tr>
<td>BULK:</td>
<td>X</td>
</tr>
<tr>
<td>SUPERSACK:</td>
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</tr>
<tr>
<td>BAG:</td>
<td></td>
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</table>

<table>
<thead>
<tr>
<th>CHEMICAL TESTS</th>
<th>ANALYSIS</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>95.98 %</td>
</tr>
<tr>
<td>SO₃</td>
<td>0.20 %</td>
</tr>
<tr>
<td>CL⁻</td>
<td>0.07 %</td>
</tr>
<tr>
<td>Total Alkali</td>
<td>0.34 %</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>0.07 %</td>
</tr>
<tr>
<td>Loss on Ignition</td>
<td>2.55 %</td>
</tr>
<tr>
<td>pH</td>
<td>6.90</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PHYSICAL TESTS</th>
<th>ANALYSIS</th>
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</thead>
<tbody>
<tr>
<td>Oversize - % retained on 45 μm sieve (wet sieved)</td>
<td>1.00 %</td>
</tr>
<tr>
<td>Density - (specific gravity)</td>
<td>2.25</td>
</tr>
<tr>
<td>Bulk Density - (per ASTM)</td>
<td>729.87 kg/m³</td>
</tr>
<tr>
<td>Specific Surface Area (by BET)</td>
<td>45.56 lb/ft³</td>
</tr>
<tr>
<td>Accelerated Pozzolanic Activity Index - with Portland Cement</td>
<td>22.95 m²/g</td>
</tr>
<tr>
<td></td>
<td>146.30 %</td>
</tr>
</tbody>
</table>

Prepared by Norchem Inc. QC Department

- 985 SEAWAY DRIVE, STE A, FORT PIERCE, FL 34949-2744
- 960 WHEELER ROAD #5357, HAUPPAUGE, N.Y. 11788
- TEL. (772) 668-6110 - FAX (772) 668-8702
- TEL. (631) 724-8639 - FAX (808) 617-8520

www.norchem.com
MasterGlenium® 7920
High-Range Water-Reducing Admixture

Description
MasterGlenium 7920 ready-to-use high-range water-reducing admixture is based on the next generation of polycarboxylate technology. This technology incorporates state-of-the-art molecular engineering to provide fast wet-out of powder materials. MasterGlenium 7920 admixture is effective in improving the day-to-day production efficiency of a concrete plant by rapidly dispersing powder materials in concrete mixtures, thereby minimizing mixing time. It is formulated to meet ASTM C 494 requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures.

Applications
Recommended for use in:
- Concrete requiring high-early compressive strength development
- Concrete mixtures with low w/cm and/or high powder contents
- Production of self-consolidating concrete (SCC) mixtures
- Green Sense® Concrete

Features
- Dosage flexibility
- Rapid cement dispersion
- Superior early and ultimate strengths

Benefits
- Optimized mixture costs
- Shorter mixing time in central mixers
- Increased productivity
- Greater batch-to-batch consistency

Guidelines for Use
Dosage: MasterGlenium 7920 admixture has a recommended dosage range of 2-12 fl oz/cwt (130-760 mL/100 kg) of cementitious materials. For most applications, dosages in the range of 2-8 fl oz/cwt (130-520 mL/100 kg) will provide excellent performance. For very high performance and SCC mixtures, up to 12 fl oz/cwt (780 mL/100 kg) of cementitious materials can be utilized. Because of variations in concrete materials, job site conditions and/or applications, dosages outside of this range may be required. In such cases, contact your local sales representative.

Mixing: MasterGlenium 7920 admixture can be added with the initial batch water or as a delayed addition. However, optimum water reduction is generally obtained with a delayed addition.

Product Notes
Corrosivity – Non-Chloride, Non-Corrosive: MasterGlenium 7920 admixture will neither initiate nor promote corrosion of reinforcing steel embedded in concrete, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of MasterGlenium 7920 admixture.

Compatibility: MasterGlenium 7920 admixture is compatible with most admixtures used in the production of quality concrete, including normal, mid- and high-range water reducers, accelerators, retarders, extended-set control admixtures, air entrainers, corrosion inhibitors, and shrinkage reducers. Do not use MasterGlenium 7920 admixture with admixtures containing beta-naphthalene sulfonate. Erratic behaviors in slump, workability retention and pumpability may be experienced.
Storage and Handling

Storage Temperature: MasterGlenium 7920 admixture must be stored at temperatures above 40 °F (5 °C). If MasterGlenium 7920 admixture freezes, thaw and reconstitute by mechanical agitation. Do not use pressurized air for agitation.

Shelf Life: MasterGlenium 7920 admixture has a minimum shelf life of 6 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of MasterGlenium 7920 admixture has been exceeded.

Packaging

MasterGlenium 7920 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Safety Data Sheets: MasterGlenium 7920 admixture

Additional Information

For additional information on MasterGlenium 7920 admixture or on its use in developing concrete mixtures with special performance characteristics, contact your BASF representative.

The Admixture Systems business of BASF’s Construction Chemicals division is the leading provider of solutions that improve placement, pumping, finishing, appearance and performance characteristics of specialty concrete used in the ready-mixed, precast, manufactured concrete products, underground construction and paving markets. For over 100 years we have offered reliable products and innovative technologies, and through the Master Builders Solutions brand, we are connected globally with experts from many fields to provide sustainable solutions for the construction industry.

Limited Warranty Notice

BASF warrants this product to be free from manufacturing defects and to meet the technical properties on the current Technical Data Guide, if used as directed within shelf life. Satisfactory results depend not only on quality products but also on use beyond our control. BASF MAKES NO OTHER WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE WITH RESPECT TO ITS PRODUCTS. The sole and exclusive remedy of Purchaser for any claim concerning this product, including but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is shipment to purchaser of product equal to the amount of product that fails to meet this warranty, at the sole option of BASF. Any claims concerning this product must be received in writing within one (1) year from the date of shipment and any claims not presented within that period are waived by Purchaser. BASF WILL NOT BE RESPONSIBLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDING LOSS PROFITS) OR PUNITIVE DAMAGES OF ANY KIND.

Purchaser must determine the suitability of the products for the intended use and assumes all risks and liabilities in connection therewith. This information and all further technical advice are based on BASF’s present knowledge and experience. However, BASF assumes no liability for providing such information and advice including the extent to which such information and advice may relate to existing third party intellectual property rights, especially patent rights, nor shall any legal relationship be created by or arise from the provision of such information and advice. BASF reserves the right to make any changes according to technological progress or further developments. The Purchaser of the Product(s) must test the product(s) for suitability for the intended application and purpose before proceeding with a full application of the product(s). Performance of the product described herein should be verified by testing and carried out by qualified experts.
V-MAR® F100

Concrete rheology-modifying admixture ASTM C494 Type S

Product Description

V-MAR® F100 is a high efficiency, rheology-modifying liquid admixture. The use of V-MAR® F100 admixture imparts lubricity to the concrete resulting in increased productivity and concrete with improved surface texture.

V-MAR® F100 admixture is supplied as a ready to use liquid that weighs approximately 8.5 lbs/gal (1.02 kg/L). V-MAR® F100 admixture does not contain intentionally added chlorides.

Uses

V-MAR® F100 is a multi-purpose admixture that reduces friction within the concrete mixture, resulting in a highly workable mixture.

V-MAR® F100 can be used for conventional slump concrete and SCC mixtures. It is particularly effective in zero slump and low slump applications such as concrete pipe, concrete extrusion, concrete paving, slip-formed concrete and roller-compacted concrete.

Advantages

Concrete produced with V-MAR® F100 offers the following advantages:

- Increased productivity through higher throughput
- Concrete moves easier and faster through machinery
- Improved paste creaminess and enhanced finishability
- Concrete consolidates with reduced vibration
- Provides superior water tolerance to the concrete making it less susceptible to normal manufacturing moisture fluctuations
- Facilitates the use of angular aggregates and/or manufactured sands in concrete
- Produces finishes with a noticeable reduction in surface defects
- Concrete requires less cement to close surfaces, resulting in lower material costs

Product Advantages

- Modifies concrete rheological properties for improved workability
- Produces concrete mixes that are cohesive without being sticky
- Facilitates concrete extrusion
- Enhanced concrete surface appearance
- Faster concrete discharge rates
Addition Rates

V-MAR® F100 is an easy to dispense liquid admixture. Dosage rates can be adjusted to meet a wide spectrum of concrete performance requirements. Addition rates for V-MAR® F100 can vary with the type of application, but will normally range from 3–12 fl oz/100 lbs (195–780 mL/100 kg). In most cases, the addition of 5–8 fl oz/100 lbs (325–520 mL/100 kg) of cementitious material will be sufficient. Please consult your GCP Applied Technologies representative for assistance with developing mix designs.

Compatibility with Other Admixtures and Batch Sequencing

V-MAR® F100 is compatible with most GCP admixtures as long as they are added separately to the concrete mix. V-MAR® F100 should be added to the concrete mix as early as possible in the batch sequence for optimum performance. Different sequencing may be used if local testing shows better performance. Please see GCP Technical Bulletin TB-0110, Admixture Dispenser Discharge Line Location and Sequencing for Concrete Batching Operations for further recommendations.

Pretesting of the concrete mix should be performed prior to use and as job conditions and materials change, in order to ensure compatibility with other admixtures, and to optimize dosage rates and addition times in the batch sequencing, in order to optimize concrete performance.

For concrete that requires air entrainment, the use of an ASTM C260 air-entraining agent is recommended to provide suitable air void parameters for freeze-thaw resistance. Please consult your GCP Applied Technologies representative for guidance.

Packaging & Handling

V-MAR® F100 is available in bulk, in totes, and drums. V-MAR® F100 will freeze at approximately 28°F (-2 °C) but will return to full functionality after thawing and thorough mechanical agitation.

Dispensing Equipment

A complete line of accurate, automatic dispensing equipment is available.
Detail Gradation Statistical Summary Report

Plant: Independent North Mine (FDOT Pit #11-490)-1050
Product: Mason Sand-1008
Specification
Period: 10/05/2017 - 10/12/2017

<table>
<thead>
<tr>
<th>Sieve/Test</th>
<th>Tests</th>
<th>Average</th>
<th>Min</th>
<th>Max</th>
<th>Range</th>
<th>St Dev</th>
<th>Target</th>
<th>Specification</th>
<th>PWS</th>
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</thead>
<tbody>
<tr>
<td>#4 (4.75mm)</td>
<td>5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<td></td>
</tr>
<tr>
<td>#8 (2.36mm)</td>
<td>5</td>
<td>0</td>
<td>0</td>
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<td>0</td>
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<td>#16 (1.18mm)</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#30 (0.8mm)</td>
<td>5</td>
<td>2</td>
<td>1</td>
<td>3</td>
<td>2</td>
<td>0.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#50 (0.3mm)</td>
<td>5</td>
<td>39</td>
<td>30</td>
<td>51</td>
<td>21</td>
<td>7.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#100 (0.15mm)</td>
<td>5</td>
<td>96</td>
<td>94</td>
<td>97</td>
<td>3</td>
<td>1.3</td>
<td></td>
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<tr>
<td>#200 (75μm)</td>
<td>5</td>
<td>99.9</td>
<td>99.8</td>
<td>100.0</td>
<td>0.2</td>
<td>0.08</td>
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<tr>
<td>Pan</td>
<td>5</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Moisture</td>
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<td>4.6</td>
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<td>FM</td>
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<td>1.38</td>
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<td>1.50</td>
<td>0.24</td>
<td>0.086</td>
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</tr>
</tbody>
</table>

Comments
Query: Query Selections
Date Created: 10/12/2017
Date Range: 10/05/2017 - 10/12/2017
Plant: Independent North Mine (FDOT Pit #11-490)-1050
Sample Type: Shipping

StonemontQC 10/12/2017 E. R. Jahn Industries, Inc.
C.4. Precaster D
# Mill Test Certificate Report - White Cement

**Types:** I/II per ASTM C150 and AASHTO M85  
**Shipping Date:** August 22, 2018  
**Cargo #:** 190

## Certification
We certify the cement described here, at the time of shipment, meets chemical and physical requirements of the current AASHTO M85 and ASTM C150 specifications for Types I and II. We further certify it meets the AASHTO M85 and ASTM C150 Optional Requirement for low alkali cement having less than 0.6% Equivalent Alkalis (Na2O + 0.658K2O). We are not responsible for improper use or workmanship.

## General Information

<table>
<thead>
<tr>
<th>Supplier</th>
<th>Lehigh White Cement Company</th>
</tr>
</thead>
<tbody>
<tr>
<td>Address</td>
<td>3920 Pendola Point Road</td>
</tr>
<tr>
<td></td>
<td>Tampa, FL 33619</td>
</tr>
</tbody>
</table>

| Source Location     | Aalborg Plant               |
|                     | Address:                    |

## Test Data on ASTM "Standard" Requirements and Results

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit Limit</th>
<th>Results</th>
<th>Item</th>
<th>Unit Limit</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>%</td>
<td>24.4</td>
<td>Fineness</td>
<td>%</td>
<td>99.3</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>%</td>
<td>6.0 max</td>
<td>2.1</td>
<td>Passing 45μm (No. 325)</td>
<td>%</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>%</td>
<td>6.0 max</td>
<td>0.3</td>
<td>Blaine Fineness</td>
<td>m²/kg</td>
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<tr>
<td>CaO</td>
<td>%</td>
<td>6.0 max</td>
<td>68.9</td>
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<td></td>
</tr>
<tr>
<td>MgO</td>
<td>%</td>
<td>6.0 max</td>
<td>0.7</td>
<td>Autoclave Expansion</td>
<td>%</td>
</tr>
<tr>
<td>SO₃</td>
<td>%</td>
<td>3.0 max</td>
<td>2.1</td>
<td>Final Time of Setting: Vicat test</td>
<td>minutes</td>
</tr>
<tr>
<td>Equivalent Alkalis</td>
<td>%</td>
<td>0.60 max</td>
<td>0.24</td>
<td>Air Content</td>
<td>%</td>
</tr>
<tr>
<td>Loss on ignition</td>
<td>%</td>
<td>3.0 max</td>
<td>1.23</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inclucible Residue</td>
<td>%</td>
<td>1.5 max</td>
<td>0.01</td>
<td>Time of setting: Vicat test</td>
<td>minutes</td>
</tr>
<tr>
<td>CO₂</td>
<td>%</td>
<td>3.0 max</td>
<td>0.01</td>
<td>Time of setting: Vicat test</td>
<td>minutes</td>
</tr>
<tr>
<td>Limestone</td>
<td>%</td>
<td>5.0 max</td>
<td>None</td>
<td>Final Time of Setting: Vicat test</td>
<td>minutes</td>
</tr>
<tr>
<td>CaCO₃ in Limestone</td>
<td>%</td>
<td>0.70 min</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Compressive Strengths

<table>
<thead>
<tr>
<th>Potential Compounds</th>
<th>% Limit</th>
<th>Results</th>
<th>Compressive Strengths</th>
<th>Limit</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>C₆S</td>
<td>75</td>
<td></td>
<td>1-Day</td>
<td>20.4</td>
<td>2951 psi</td>
</tr>
<tr>
<td>C₆S</td>
<td>13</td>
<td></td>
<td>3-Day</td>
<td>10.0</td>
<td>1492 psi</td>
</tr>
<tr>
<td>C₆AF</td>
<td>8 max</td>
<td>5</td>
<td>7-Day</td>
<td>17.0</td>
<td>2525 psi</td>
</tr>
<tr>
<td>C₆AF</td>
<td>1</td>
<td></td>
<td>28-Day</td>
<td>48.5</td>
<td>7033 psi</td>
</tr>
</tbody>
</table>

---

*Not Applicable.*  
*Limit applies only to Type II cement. Not Applicable for Type I.*  
*Optional % Limit for Equivalent Alkalis (Na₂O + 0.658K₂O)*  
*1 MPa = 145 psi*

---

**Date:** September 25, 2018  
**Niels Lundgaard**

---

eConstruct-WJE-UNL-NCSU 293 January 20
**RUSSTECH CSF**

**CONDENSED SILICA FUME
ADMIXTURE FOR CONCRETE**

**DESCRIPTION:**
RUSSTECH CSF is manufactured from the highest grade silica fume and is used to produce ultra-high strength, high durability concrete.

RUSSTECH CSF consists of highly reactive ultra-fine particles and contains 92% to 97% silicon dioxide and has an average specific gravity of 2.20. Other silica fumes, with lower amounts of silicon dioxide, may contain other unwanted elements and compounds such as carbon. Higher levels of carbon, as indicated by loss on ignition, darken concrete and can lead to problems with entrained air.

**ADVANTAGES:**
- Improves the relative durability of concrete by lowering the permeability of the concrete significantly.
- Increases early and ultimate strengths both compressive and flexural
- Ultra-high strength concrete exceeding compressive strengths of 15,000 psi and 6,000,000 psi modulus of elasticity
- Increases aggressive chemical resistance up to 15 times that of normal concrete
- Increases density
- Reduces damage caused by freezing and thawing
- Increases the abrasion resistance of the concrete
- Reduces surface bleeding
- Effective corrosion inhibiting system that resists the ingress of water-borne salt contaminants (de-icing salts or marine salts)
- Reduces segregation

**HOW THE PRODUCT WORKS:**
RUSSTECH CSF is a highly reactive pozzolan that reacts with the cement and converts weak calcium hydroxide crystals into useful calcium silicate hydrate gel.
The abundance of ultra-fine silica fume places thousands of particles for each cement grain in a typical concrete mixture.
The sub-micron particles are dispersed in spaces around and between the cement grains, leading to uniform distribution of particle sizes and high density concrete.

<table>
<thead>
<tr>
<th>AASHTO T-277 Chloride Permeability Test Based On Electrical Charge Passed</th>
<th>Chloride Permeability</th>
<th>W/C Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Charge Passed (Coulombs)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;4000</td>
<td>High</td>
<td>&gt;0.60</td>
</tr>
<tr>
<td>2000-4000</td>
<td>Moderate</td>
<td>0.40-0.50</td>
</tr>
<tr>
<td>1000-2000</td>
<td>Low</td>
<td>&lt;0.40</td>
</tr>
<tr>
<td>100-1000</td>
<td>Very Low</td>
<td>Latex</td>
</tr>
<tr>
<td>&lt;100</td>
<td>Negligible</td>
<td>Polymer</td>
</tr>
</tbody>
</table>

RUSSTECH CSF significantly reduces the permeability of the concrete and increases the compressive strength.

**SPECIFICATIONS:**
COMPLEXITY:
RUSSTECH CSF is compatible with all types of Portland cement, class C and F flyash, fibers, air entraining, water reducing, corrosion-inhibitors and superplasticizing admixtures. For best results, each admixture must be introduced separately into the concrete mix.

STORAGE/HANDLING:
RUSSTECH CSF stores, handles, and batches similar to Portland cement. Bulk storage can be in a normal silo. Consult your RussTech technical service representative for proper procedures for discharging and optimum storage silo conditions with bulk delivery.

PACKAGING:
RUSSTECH CSF is available in bulk delivery, bulk bags, and 25 lb. or 50 lb. shredable bags.

SHELF LIFE:
RUSSTECH CSF, packaged in bags, can be stored indefinitely provided it is kept in a dry environment.

VISIT US ON THE WEB AT:
www.RussTechnet.com

USES:
RUSSTECH CSF can be used to consistently produce high durability and high strength concrete wherever this application is necessary. Some examples where dramatically reduced permeability and additional protection from corrosion (de-icing salts) are necessary, would be parking garages, bridges, high early strength concrete, Precast, Prestressed applications. These high performance characteristics enable concrete to attain ultra-high strength and extended service years of the concrete.

RUSSTECH CSF enhances the durability of the concrete when in the presence of aggressive chemical attack, also, extending the service years of the concrete in harsh environments.

TECHNICAL NOTE:
RUSSTECH CSF does not contain calcium chloride or any chloride-based components. It will not promote or contribute to corrosion of reinforcing steel in concrete.

DOSAGE RATE:
RUSSTECH CSF is recommended for use at a rate of 5 to 15% by weight of the cement. Dosage rates may vary depending on the application and desired concrete properties.

Because local job conditions vary, contact your local RussTech technical service representative for further assistance if using this product outside recommended dosage ranges or when combining with other admixtures.

11208 Decimal Drive  Louisville, KY 40299  Phone: (502) 267-7700  Fax: (502) 267-8922
### Customer
**Precaster D**
USA

### Other Contacts
Kenney Alderman  
Carmeuse Lime & Stone  
Filler Products Operation  
Chatsworth, GA 30705  
kenney.alderman@carmeusena.com

---

**Inspection lot:** 8900002522734  
**GFP:** 60C  
**Plant:** Chatsworth  
**Monthly Averages For January 2018**  
**Sampling Date:** 02/01/2018  
**Printed on:** 02/06/2018

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Unit</th>
<th>Value</th>
<th>Lower Limit</th>
<th>Upper Limit</th>
</tr>
</thead>
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<tr>
<td>-325M</td>
<td>%</td>
<td>56.1</td>
<td>52.0</td>
<td>60.0</td>
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*** End ***
# Typical Analysis

## #55 Sand/X-Fine Sand

Lugoff Plant, South Carolina

<table>
<thead>
<tr>
<th>mm</th>
<th>US Mesh</th>
<th>% Passing Cumulative</th>
<th>% Retained Per Sieve</th>
<th>Anchor Pattern</th>
<th>Material Uses</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.060</td>
<td>8</td>
<td>100.00</td>
<td>0.00</td>
<td>0.5 - 1.0</td>
<td>High pressure</td>
</tr>
<tr>
<td>1.180</td>
<td>16</td>
<td>100.00</td>
<td>0.00</td>
<td></td>
<td>blast system, smooth surfaces, trucks also used for play sand</td>
</tr>
<tr>
<td>0.850</td>
<td>20</td>
<td>99.88</td>
<td>0.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.600</td>
<td>30</td>
<td>98.20</td>
<td>1.68</td>
<td></td>
<td></td>
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<tr>
<td>0.425</td>
<td>40</td>
<td>89.77</td>
<td>8.43</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.300</td>
<td>50</td>
<td>66.61</td>
<td>23.16</td>
<td>Min Size Nozzle</td>
<td></td>
</tr>
<tr>
<td>0.212</td>
<td>70</td>
<td>35.49</td>
<td>31.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.150</td>
<td>100</td>
<td>12.08</td>
<td>23.41</td>
<td>1/8&quot;</td>
<td></td>
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<tr>
<td>0.106</td>
<td>140</td>
<td>1.99</td>
<td>10.09</td>
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<tr>
<td>0.090</td>
<td>200</td>
<td>0.21</td>
<td>1.79</td>
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<tr>
<td>0.053</td>
<td>270</td>
<td>0.03</td>
<td>0.18</td>
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<tr>
<td>&lt;0.053</td>
<td>PAN</td>
<td>0.00</td>
<td>0.03</td>
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</table>
SUPERFLO 2040 RM

MULTI-RANGE
SUPER PLASTICIZING
ADMIXTURE FOR CONCRETE

DESCRIPTION:
SUPERFLO 2040 RM is a normal setting multi-range water-reducing admixture for concrete utilizing polycarboxylate technology. It is designed to facilitate the placing and finishing of ready-mixed concrete that is highly flowable and workable for extended periods of time with normal setting characteristics.

ADVANTAGES:
- By varying the dosage rate, Superflo 2040 RM can be used as a normal, mid-range and high-range water-reducing admixture
- Improves the quality of concrete by decreasing water-cementitious ratio
- Increases high early and ultimate strength both compressive and flexural
- High durability and increased density
- Reduces damage caused by freezing and thawing
- Reduces water content needed for a given workability (12-40%)
- Reduces surface bleeding
- Reduces cracking, creep, and shrinkage
- Reduces segregation and increase cohesiveness
- Improves finishability and workability of concrete
- Improves pumpability of concrete
- Maintains slump life during extended mixing times
- Plasticity range of 8 to 11 inches
- Improves bond strength to the steel
- Ideally suited for use in self-consolidating concrete mixes
- Higher productivity and reduced labor

DOSAGE RATE:
SUPERFLO 2040 RM is recommended for use at a dose of 1 to 8 fluid ounces per 100 pounds (65 to 522 mL per 100 kg) of cementitious to meet the requirements of ASTM C 494 Type A water-reducing and Type F high-range water-reducing admixture.

SPECIFICATIONS:
Conforms to:
ASTM C 494 Types A and F
AASHTO M 194 Types A and F
CRD C 87 Types A and F
All other Federal and State specifications

Flowing concrete with no segregation can be obtained using SUPERFLO 2040 RM along with proper mix design practices for self-consolidating concrete.
**TECHNICAL NOTE:**

SUPERFLO 2040 RM does not contain calcium chloride or any chloride-based components. It will not promote or contribute to corrosion of reinforcing steel in concrete.

SUPERFLO 2040 RM conforms to the minimum chloride ion limits published by current construction industry standards.

**DIRECTIONS:**

SUPERFLO 2040 RM should be added with the initial mixing water or incorporated with the final water at the end of the batch sequence. It is not unusual to experience significantly lower air entrainment dosage requirements (50-75%) when compared to conventional high-range water reducers.

**STORAGE:**

SUPERFLO 2040 RM may freeze at temperatures below 35 F (2 C). Although freezing does not harm SUPERFLO 2040 RM, precautions should be taken to protect it from freezing. If it should freeze, thaw at 45 F and reconstitute with mechanical agitation. **Do Not Use Pressurized Air For Agitation.**

**COMPATIBILITY:**

SUPERFLO 2040 RM is compatible with all types Portland cement, class C and F fly ash, silica fume, fibers, approved air entraining, and water-reducing admixtures.

SUPERFLO 2040 RM can be used in white, colored, and architectural concrete. For best results, each admixture must be introduced separately into the concrete mix.

**PACKAGING:**

5-gallon pails, 55-gallon drums and 275-gallon totes all non-returnable containers.

**SHELF LIFE:**

18 months

**VISIT US ON THE WEB AT:**

www.RussTechnet.com

---

EASE OF PLACEMENT:

11208 Decimal Drive Louisville, KY 40299 Phone: (502) 267-7700 Fax: (502) 267-8922
EXTENDFLO X90

SPECIALTY SLUMP RETAINING ADMIXTURE FOR CONCRETE

DESCRIPTION:
EXTENDFLO X90 is a revolutionary new technology based on significant advances in the admixture industry. When used as part of an admixture system, it provides slump retention without retardation. EXTENDFLO X90 gives the concrete producer the ability to immediately create the ideal admixture system for fluctuating regional raw materials, environmental conditions and project requirements.

Recommended for use in:
- Concrete with varying slump requirements
- Concrete mixes utilizing supplementary cementitious materials
- Concrete when increased stability, durability and high flow-ability are required
- Production of self-consolidating concrete (SCC) mixes

ADVANTAGES:
- Workability retention without retardation
- Flexible levels of workability retention by adjusting dosage
- Improved early and late age compressive strengths
- Promotes greater consistency of concrete workability at the jobsite
- Promotes consistency in compressive strengths via minimized jobsite addition of water
- Minimizes re-dosing of high range water-reducing admixture at the jobsite
- Consistent air entrainment
- Fewer rejected loads and better customer satisfaction due to consistent quality of concrete

SPECIFICATIONS:
Conforms to:
ASTM C 494 Type A
AASHTO M 194 Type A

DOSEAGE RATE:
EXTENDFLO X90 admixture has a recommended dosage range of 2 to 12 fl. oz. per 100 lbs. of cementitious material. It can be added with the initial batch water or as a delayed addition.

TECHNICAL NOTE:
EXTENDFLO X90 admixture will neither initiate nor promote corrosion of reinforcing steel embedded in concrete, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of this product. A technical representative can provide additional info on EXTENDFLO X90 and its use in developing concrete mixes with special performance characteristics.

COMPATABILITY:
EXTENDFLO X90 admixture is compatible with most admixtures used in the production of quality concrete including normal, mid-range and high-range water reducing admixtures, air entrainers, accelerators, retarders, extended set control admixtures, corrosion inhibitors and shrinkage reducers. EXTENDFLO X90 is not recommended for use with any Poly Naphthalene Sulfonate based admixtures.
STORAGE:
EXTENDFLO X90 may freeze at temperatures below 35 F (2 C). Although freezing does not harm EXTENDFLO X90, precautions should be taken to protect it from freezing. If it should freeze, thaw at 45 F and reconstitute with mechanical agitation. Do Not Use Pressurized Air For Agitation.

PACKAGING:
5-gallon pails, 55-gallon drums and 275-gallon totes all non-returnable containers.

SHELF LIFE:
18 months

VISIT US ON THE WEB AT:
www.RussTechnet.com
AIR DETRAINER-1

AIR DETRAINING
ADMIXTURE FOR CONCRETE

DESCRIPTION:
Air Detainer-1 is for plastic concrete. It helps eliminate foaming and minimizes air entrainment in cement slurries, grouts, concrete and mortars. It can also be used to counteract the air entrainment caused by water reducers and super plasticizers.

ADVANTAGES:
• Reduces the amount of concrete rejected due to higher entrained air contents
• Increases unit weight of concrete
• Recommended for the production of heavyweight concrete
• Counteracts entrapped/entrained air caused by super plasticizers and water reducers
• Helps maintain higher densities
• Can be added at the plant or job site

DOSEAGE RATE:
Air Detainer-1 is for plastic concrete. It should be dosed at 2 to 16 ounces per cubic yard (130 to 1044 ml per cubic meter). An overdose will only make it harder to put air back into the concrete.

COMPATIBILITY:
Air Detainer-1 is fully effective and compatible in concrete containing all types of Portland cement, class C and F fly ash, silica fume, calcium chloride, fibers, and approved water-reducing, accelerating and retarding admixtures.

Air Detainer-1 can be used in white, colored, and architectural concrete. For best results, the air entrainment should be dispensed separately into the mix with the initial batch water or on damp, fine aggregate.

PACKAGING:
5 gallon pails and 55-gallon drums

SHELF LIFE:
18 months

VISIT US ON THE WEB AT:
www.RussTechnet.com

RusTech Inc.
“We Add The Difference”

11208 Decimal Drive  Louisville, KY 40299  Phone: (502) 267-7700  Fax: (502) 267-8922
C.5. Precaster E
ASH GROVE CEMENT COMPANY

Quantities (tons):
Trailer Car:
Shipped:

Production Period: October 2018

The following information is based on average test data during the production period. The data is typical of cement shipped from the Louisville, Kentucky plant. Individual shipments may vary.

STANDARD REQUIREMENTS
ASTM C150-37

<table>
<thead>
<tr>
<th>Item</th>
<th>A.S.T.M. Test Method</th>
<th>Spec. Limit</th>
<th>Test Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>SO₃ (%)</td>
<td>C114</td>
<td>A</td>
<td>2.0</td>
</tr>
<tr>
<td>R₂O₃ (%)</td>
<td>C114</td>
<td>A</td>
<td>4.4</td>
</tr>
<tr>
<td>Fe₂O₃ (%)</td>
<td>C114</td>
<td>A</td>
<td>1.7</td>
</tr>
<tr>
<td>CaO (%)</td>
<td>C114</td>
<td>A</td>
<td>64.0</td>
</tr>
<tr>
<td>MgO (%)</td>
<td>C114</td>
<td>0.0 max</td>
<td>2.0</td>
</tr>
<tr>
<td>SiO₂ (%)</td>
<td>C114</td>
<td>3.5 max</td>
<td>3.8</td>
</tr>
<tr>
<td>Loss on Ignition (%)</td>
<td>C114</td>
<td>3.0 max</td>
<td>1.5</td>
</tr>
<tr>
<td>Na₂O (%)</td>
<td>C114</td>
<td>A</td>
<td>0.13</td>
</tr>
<tr>
<td>K₂O (%)</td>
<td>C114</td>
<td>A</td>
<td>0.53</td>
</tr>
<tr>
<td>Insoluble Residues (%)</td>
<td>C114</td>
<td>1.5 max</td>
<td>0.52</td>
</tr>
<tr>
<td>Potential compounds (%)</td>
<td>C114</td>
<td>A</td>
<td>6.1</td>
</tr>
<tr>
<td>C₃A (%)</td>
<td>C114</td>
<td>A</td>
<td>1.3</td>
</tr>
<tr>
<td>C₃S (%)</td>
<td>C114</td>
<td>15 max</td>
<td>7</td>
</tr>
<tr>
<td>C₃AF (%)</td>
<td>C114</td>
<td>A</td>
<td>8</td>
</tr>
<tr>
<td>Air content of mortar (volume %)</td>
<td>C115</td>
<td>2</td>
<td>7</td>
</tr>
<tr>
<td>Fineness (m²/kg)</td>
<td>C234</td>
<td>A</td>
<td>480</td>
</tr>
<tr>
<td>Air permeability</td>
<td>C151</td>
<td>0.80 max</td>
<td>-0.01</td>
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<tr>
<td>Autoclave expansion (%)</td>
<td>C151</td>
<td>1.740</td>
<td>667</td>
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<tr>
<td>Compressive strength (psi)</td>
<td>C109</td>
<td>28 Days</td>
<td>4850</td>
</tr>
<tr>
<td>1 Day</td>
<td>3 Day</td>
<td>7 Day</td>
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</tr>
<tr>
<td>1740 min</td>
<td>3180 min</td>
<td>6700</td>
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<td>4850</td>
<td>6670</td>
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<td>6700</td>
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<td>Time of setting (minutes) (Vicat)</td>
<td>C191</td>
<td>20</td>
<td>55</td>
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<td>Initial: Not more than</td>
<td>C191</td>
<td>375</td>
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<tr>
<td>Not more than</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Mortar Bar Expansion</td>
<td>C1038</td>
<td>D, 8</td>
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OPTIONAL REQUIREMENTS
ASTM C150-17, Tables 2 and 4

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<tr>
<th>Item</th>
<th>A.S.T.M. Test Method</th>
<th>Spec. Limit</th>
<th>Test Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent alkalies (%)</td>
<td>C154</td>
<td>0.60</td>
<td>0.48</td>
</tr>
</tbody>
</table>

A = Not applicable.
B = Test result represents most recent value and is provided for information only.
C = Test results for this period not yet available.
D = Required only if the percent SO₃ exceeds 3.5, in which case the expansion shall not exceed 0.002% at 14 days.

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirements of ASTM C150/C150M-17 (Type III) and AASHTO MBS-16 (Type III), or (other) specification.

Signature: [Signature]

Amadeo Lampe
FORCE 10,000® D

High performance concrete admixture dry densified powder

Product Description

FORCE 10,000® D is a dry densified microsilica (silica fume) powder designed to increase concrete compressive and flexural strengths, increase durability, reduce permeability and improve hydraulic abrasion-erosion resistance. The specific gravity of FORCE 10,000® D is 2.20.

Uses

FORCE 10,000® D can be used to consistently produce concrete with strengths of 6,000 psi (42 MPa) and higher in most instances with locally available materials and existing methods. It may also be used in precast and prestress applications where high early strengths are required.

The addition of FORCE 10,000® D also produces concrete with increased watertightness and dramatically reduced permeability compared to conventional mixes. Reduced permeability is an important advantage in slowing the intrusion of chloride where corrosion of reinforcing steel is a potential problem. Examples are parking garages, bridge decks and concrete in a marine environment. FORCE 10,000® D also enhances the durability of concrete against aggressive chemical attack and in hydraulic abrasion-erosion applications.

Preconstruction Trial Mix

It is strongly recommended that trial mixes be made several weeks before construction start up. This will allow the concrete producer an opportunity to determine the proper batching sequence and amounts of other admixtures needed in order to deliver the required concrete mix to the job site. A trial mix will also help determine whether the combination of concrete materials and construction practices will allow the concrete to meet a specified performance. GCP’s broad experience with this product can help the concrete producer deliver a satisfactory product regardless of the mixture proportions. Contact your GCP Applied Technologies sales representative for help with trial mixes.

Finishing & Curing

FORCE 10,000® D concrete can be used in flatwork with little or no modification to the recommended practices outlined in ACI 302, Guide for Concrete Floor and Slab Construction.

FORCE 10,000® D will reduce the surface bleed water of concrete in large applications. ACI 308, Standard Practice for Curing Concrete, must be followed to ensure that any problems that can occur due to decreased bleeding are minimized. Your GCP Applied Technologies representative is available to review your particular job needs.
Performance

FORCE 10,000® D improves concrete through two mechanisms. The extremely fine microsilica particles are able to fill the microscopic voids between the cement particles, creating a less permeable structure. In addition, the microsilica reacts with the free calcium hydroxide within the concrete to form additional calcium silicate hydrate (glue), producing a tighter paste-to-aggregate bond. FORCE 10,000® D does not affect concrete set times.

FORCE 10,000® D will improve the mechanical properties of concrete. In order to meet specified concrete performance levels, however, many variables are involved. These include, but are not limited to; concrete materials, weather conditions, testing techniques and mixing, transporting, placing and finishing practices. ACI and ASTM guidelines must be strictly adhered to.

Addition Rates

FORCE 10,000® D dosage rates will vary based on the requirements of the application. Dosage rates should be calculated on percent microsilica by weight of cement, or on lb/yd³ (kg/m³) of concrete, as appropriate. Dosage rates will be as specified. If not specified, consult your GCP Applied Technologies representative for your particular job needs.

Compatibility with Other Admixtures and Batch Sequencing

FORCE 10,000® D is compatible with all conventional water reducers, superplasticizers, set retarders and DCP® corrosion inhibitor. Any air-entraining agent which works effectively with superplasticizers and microsilica, particularly vinyl resins such as DARAVAIR® by GCP Applied Technologies, are recommended. Only non-chloride set accelerators, such as POLARSET®, may be used with FORCE 10,000® D concrete. All admixtures must be added separately to assure their prescribed performance. Trial mixes and pretesting of concrete are recommended to optimize dosage rates, and ensure ultimate performance.

FORCE 10,000® 10,000 D can be used in either central or transit mix concrete production. FORCE 10,000® D may be used in conjunction with water reducing admixtures (both normal and high-range as approved by ASTM) to assure workability of the mix.

Packaging, Handling and Storage

FORCE 10,000® D is available in bulk, and 25 lbs (11.4 kg) Concrete Ready Bags™.

Bagged FORCE 10,000® D should be stored in a dry, protected area. Manual dispensing by tearing the bags is the normal method. A dust mask should be used when dispensing the bagged product, consult the product MSDS for more complete instructions.
Dispensing Equipment

Bulk FORCE 10,000® D may be stored in already existing cement silos. The silos must be completely clean with no foreign residue remaining which may cause contamination. Up-pipes to the silo for unloading bulk tankers should also be clean and clear of obstructions. Small diameter 4 in. (100 mm) rigid metal pipes with several angles (especially right angles) will cause longer unloading times. Large diameter 6 in. (150 mm) flat lined, flexible rubber pipes will allow for the least unloading time. Dispensing bulk FORCE 10,000® D will take place in the same manner as that used for cement. Augering or dropping from the silo to the weigh hopper is the usual practice.
<table>
<thead>
<tr>
<th>Analysis</th>
<th>Value</th>
<th>ASTM C 618 CLASS C</th>
<th>AASHTO M 295 CLASS C</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2 (silicon dioxide), %</td>
<td>43.03</td>
<td>50 min</td>
<td>50 min</td>
</tr>
<tr>
<td>Al2O3 (aluminum oxide), %</td>
<td>20.00</td>
<td>5.0 max</td>
<td>5.0 max</td>
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<tr>
<td>Fe2O3 (iron oxide), %</td>
<td>5.98</td>
<td>3.0 max</td>
<td>3.0 max</td>
</tr>
<tr>
<td>SiO2+Al2O3+Fe2O3, %</td>
<td>69.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CaO (calcium oxide), %</td>
<td>21.94</td>
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<td>5.0 max</td>
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<tr>
<td>MgO (magnesium oxide), %</td>
<td>4.18</td>
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<td></td>
</tr>
<tr>
<td>SO3 (sulfur trioxide), %</td>
<td>0.88</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Moisture content, %</td>
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<td></td>
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</tr>
<tr>
<td>Loss On Ignition, %</td>
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<td></td>
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<tr>
<td>Na2O (sodium oxide), %</td>
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<td></td>
</tr>
<tr>
<td>K2O (potassium oxide), %</td>
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</tbody>
</table>

**PHYSICAL ANALYSES**

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Value</th>
<th>ASTM C 618 CLASS C</th>
<th>AASHTO M 295 CLASS C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fineness, amount retained on #325 sieve, %</td>
<td>21.6</td>
<td>34 max</td>
<td>34 max</td>
</tr>
<tr>
<td>variation, points from average</td>
<td>-2.3</td>
<td>5 max</td>
<td>5 max</td>
</tr>
<tr>
<td>Density, g/cm³</td>
<td>2.55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>variation from average, %</td>
<td>-3.3</td>
<td>5 max</td>
<td>5 max</td>
</tr>
<tr>
<td>Strength Activity Index with Portland Cement at 7 days, % of cement control</td>
<td>89</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water Requirement, % of cement control</td>
<td>94</td>
<td>105 max</td>
<td>105 max</td>
</tr>
<tr>
<td>Soundness, autoclave expansion or contraction, %</td>
<td>0.02</td>
<td>0.8 max</td>
<td>0.8 max</td>
</tr>
</tbody>
</table>

We hereby certify that the fly ash represented by the above chemical and physical analysis meets the requirements of ASTM C 618 and AASHTO M 295.

_Tina M. Tucholski_  
KCPC Laboratory Manager  
10/31/2018

**Note:** Finely divided materials may tend to reduce the entrained air content of concrete. Hence, if a mineral admixture is added to any concrete for which entrainment of air is specified, provision should be made to ensure that the specified air content is maintained by air content tests and by use of additional air-entraining admixture or use of an air-entraining admixture in combination with air-entraining hydraulic cement.
## Material Certification Report

**Brand:** Skyway Cement  
**Material:** Slag Cement  
**Grade:** 100  
**Data Range:** August 1-31, 2018  
**Lot Number:** Multiple Lots

### Certification

This cement meets the requirements of ASTM specification C989 for Grade 100 Slag Cement.

### General Information

**Supplier:** Skyway Cement Company LLC  
**Address:** 3020 East 103rd Street  
**Telephone:** (672)302-5910  
**Date Issued:** 14-Sep-18

The following information is based on average test data during the test period. The data is typical of slag cement shipped by Skyway Cement Company LLC; individual shipments may vary.

<table>
<thead>
<tr>
<th>Item</th>
<th>Limit*</th>
<th>Result</th>
<th>Physical</th>
<th>Limit*</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>48h m.s (No. 385)ake (%)</td>
<td>30 max</td>
<td>6.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blaine Fineness (m²/kg)</td>
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<td></td>
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<tr>
<td>Sulfate S (%)</td>
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<td>0.97</td>
<td></td>
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<tr>
<td>Sulfate Ion - SO₄ (%)</td>
<td>-</td>
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<td></td>
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</tr>
<tr>
<td>Slag Achility Index (%)</td>
<td>75 min</td>
<td>80</td>
<td></td>
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</tr>
<tr>
<td>Slag 28 Day Index</td>
<td>95 min</td>
<td>111</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Compressive Strength - MPa (psi):</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 Day</td>
<td></td>
<td></td>
<td></td>
<td>25</td>
<td>3605</td>
</tr>
<tr>
<td>28 Day</td>
<td></td>
<td></td>
<td></td>
<td>44</td>
<td>6470</td>
</tr>
<tr>
<td>Reference Cement</td>
<td></td>
<td></td>
<td></td>
<td>36</td>
<td>4460</td>
</tr>
<tr>
<td>7 Day</td>
<td></td>
<td></td>
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<td>46</td>
<td>6072</td>
</tr>
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<td>28 Day</td>
<td></td>
<td></td>
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</table>

### Reference Cement Qualification Data

<table>
<thead>
<tr>
<th>Item</th>
<th>Limit*</th>
<th>Result</th>
<th>Physical</th>
<th>Limit*</th>
<th>Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 Day</td>
<td>50.6</td>
<td>0.93</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C₃S</td>
<td>-</td>
<td>0.78</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C₃S</td>
<td>-</td>
<td>57.8</td>
<td></td>
<td></td>
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<tr>
<td>C₅A</td>
<td>-</td>
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<td></td>
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<tr>
<td>C₄AF</td>
<td>-</td>
<td>8.2</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>28 Day</td>
<td>34.8</td>
<td>0.93</td>
<td></td>
<td>36.9</td>
<td>5026</td>
</tr>
</tbody>
</table>

### Notes

*All values in the limit columns means Not Applicable.

**Reference cement results have produced “Preparation of Specimens”, Information on Reference Cement certification available upon request.

Specific Gravity: 2.68

This date may have been reported on previous mill certificates. It is typical of the cement being currently shipped which was produced in August of 2018.
### Sieve Analysis

**Material:** #10 SAND  
**Specification:** Plant Spec

<table>
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<tr>
<th>Sieve Size</th>
<th>#8</th>
<th>#16</th>
<th>#30</th>
<th>#50</th>
<th>#100</th>
<th>#200</th>
<th>Pan</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3.66</td>
<td>0.6%</td>
<td>3.6%</td>
<td>30.8%</td>
<td>89.8%</td>
<td>22.4%</td>
<td>21.6%</td>
<td>2.4%</td>
<td>569.0%</td>
</tr>
<tr>
<td>#2.4</td>
<td>0.6%</td>
<td>0.6%</td>
<td>15.6%</td>
<td>57.8%</td>
<td>57.7%</td>
<td>38.4%</td>
<td>2.4%</td>
<td>569.0%</td>
</tr>
<tr>
<td>#1.178</td>
<td>2.4%</td>
<td>3.1%</td>
<td>38.9%</td>
<td>99.1%</td>
<td>96.1%</td>
<td>99.6%</td>
<td>0.4%</td>
<td>100.0%</td>
</tr>
<tr>
<td>#0.65</td>
<td>2.4%</td>
<td>3.1%</td>
<td>38.9%</td>
<td>99.1%</td>
<td>96.1%</td>
<td>99.6%</td>
<td>0.4%</td>
<td>100.0%</td>
</tr>
</tbody>
</table>

**Date:** 11/13/2018

**Specified Limits:**
- % Retained: 65-100%
- % Passing: 10-40%
Appendix C – Precaster Materials – Mill Reports

HRWR – High Cementitious Content Concrete

- Features
  CHRYSO® Fluid Premia 150 is a new generation high range water reducing admixture based on modified polycarboxylates.

  CHRYSO® Fluid Premia 150 is formulated specifically to allow for manufacturing of very high cementitious content concrete and achieve superior strength performance at all ages.

  CHRYSO® Fluid Premia 150 exclusive formulation allows for extreme easiness of use and robustness.

  CHRYSO® Fluid Premia 150 is manufactured under rigid quality control standards to provide uniform, reliable results.

- Benefits
  - Provides enhanced workability retention
  - Provides increased slump and flowability without increased water content
  - Improves finish and placement of concrete
  - Allows for high strength performance at all ages
  - Improves concrete quality by reducing the water-cement ratio for a given degree of workability
  - Proprietary molecule allows for easiness of use and concrete performance consistency
  - Reduces cracking and shrinkage
  - Improves concrete chemical resistance and durability
  - Improves cementitious material performance (more psf/lb)

- Areas of Application
  CHRYSO® Fluid Premia 150 is recommended for all concrete mixes where significant water reduction, improved cementitious material performance (more psf/lb), improved finishing, good slump retention and enhanced flowability characteristics are desirable including SCC.

  CHRYSO® Fluid Premia 150 is especially recommended for use in Ultra High Performance Concrete applications where high flowability characteristics, high strengths at all ages and extended workability are required.

![Compressive strengths lbs/in² vs Time (Days)](chart)

CHRYSO® Fluid Premia 150

www.chrysoinc.com
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix C – Precaster Materials – Mill Reports

CHRYSO® Fluid Premia 150

Description:

- Characteristics:
  - Physical state: liquid
  - Density: Approx. 1.06
  - pH: Approx. 5
  - Chloride content: Nil

CHRYSO® Fluid Premia 150 does not contain any purposely added calcium chloride or other chloride-based components. It will not promote or contribute to corrosion of reinforcing steel in concrete.

Packaging:
- 55 gallon (210 L) drums
- 264 gallon (1000 L) totes
- Bulk deliveries

Standard specifications:
- Conforms to ASTM C 494 Type A & F
- AASHTO M 194 Type A & F
- GB 887 Type A & F

Directions for use:

Dosage:
CHRYSO® Fluid Premia 150 is recommended for use at a dosage rate of 3 to 6 fluid ounces per 100 pounds (195 to 391 ml per 100 kg) of cementitious material for a Type A and 3 to 4 fluid ounces per 100 pounds (232 to 260 ml per 100 kg) of cementitious material for a Type F.

CHRYSO® Fluid Premia 150 can be added at the concrete plant with the initial or tail water or on the job site. In case of addition in a mixing truck, it is recommended that the concrete be mixed at high speed for approximately 3 – 5 minutes.

Because local job conditions vary, please contact your local CHRYSO sales representative for further assistance if using outside recommended dosage ranges.

Compatibility
CHRYSO® Fluid Premia 150 is compatible with all types of Portland cement, class C and F fly ash, slag, microsilica, calcium chloride, fibers and approved air entraining admixtures.

CHRYSO® Fluid Premia 150 can be used in all white, colored, and architectural concrete. For best results, each admixture must be dispensed separately into the concrete mix.

Precaution:
CHRYSO® Fluid Premia 150 may freeze at temperatures below 28°F (-2°C). Although freezing does not harm CHRYSO® Fluid Premia 150, precautions should be taken to protect it from freezing.

If CHRYSO® Fluid Premia 150 should happen to freeze, thaw and reconstitute with mechanical agitation.

Do not store the product at temperatures above 100°F (38°C) or under 40°F (5°C) for long periods.

Shelf life: 9 months.

Safety:
CHRYSO® Fluid Premia 150 is not considered dangerous to handle. Please refer to the material safety data sheet for additional information.

About CHRYSO:
A worldwide leader for Concrete and Cement additives, CHRYSO has been servicing the construction industry for over half a century with outstanding innovation and service. As a result, CHRYSO’s name and products have been associated with the most prestigious and demanding construction projects worldwide.

CHRYSO INC Tel: (800) 936-7552 – 972-772-0010
Southern Division P.O. Box 190 Rockwall, TX 75032
Midwest Division P.O. Box 129 Charlestown, IN 47111
Western Division 5090 Norse St Denver, CO 80239

www.chrysoinc.com

This information contained in this document is given to the best of our knowledge and to the best of our ability and belief. However, it cannot be considered a warranty, certification or guarantee. Users must perform tests on the product in their facility to verify that the methods and conditions of use of the product are satisfactory. Our specialists remain at the disposal of customers if they require help with the application of the product for their specific needs.
**D. APPENDIX D - PRECASTER MATERIAL CHARACTERIZATION**

**D.1. Precaster A**

*Figure D.1-1. Particle Size Gradations for Precaster A - Solid Materials. *Silica Fume Gradation Assumed from Literature Sources (Elkem Materials 2016)*

*Figure D.1-2. Andreasen and Andersen (A&A) Target Gradation and Combined Gradation for Recommended Mix Design for Precaster A. Error for Final Mix = 153*
D.2. Precaster B

Figure D.2-1. Particle Size Gradations for Precaster B - Solid Materials. *Silica Fume Gradation Assumed from Literature Sources (Elkem Materials 2016)

Figure D.2-2. Andreasen and Andersen (A&A) Target Gradation and Combined Gradation for Recommended Mix Design for Precaster B Error for Final Mix = 112
D.3. Precaster C

**Figure D.3-1. Particle Size Gradations - Solid Materials for Precaster C.** *Silica Fume Gradation Assumed from Literature Sources (Elkem Materials 2016)*

**Figure D.3-2. Andreasen and Andersen (A&A) Target Gradation and Combined Gradation for Recommended Mix Design for Precaster C. Error for Final Mix = 210*
D.4. Precaster D

Figure D.4-1. Particle Size Gradations - Solid Materials for Precaster D Silica Fume Gradation Assumed from Literature Sources (Elkem Materials 2016)

Figure D.4-2. Andreasen and Andersen (A&A) Target Gradation and Combined Gradation for Recommended Mix Design for Precaster D. Error for Final Mix = 170
D.5. Precaster E

**Figure D.5-1. Particle Size Gradations - Solid Materials for Precaster E.** *Silica Fume Gradation Assumed from Literature Sources (Elkem Materials 2016)*

**Figure D.5-21. Andreasen and Andersen (A&A) Target Gradation and Combined Gradation for Recommended Mix Design for Precaster E. Error for Final Mix = 154*
E. APPENDIX E - DESIGN EXAMPLES OF BUILDING FLOOR SYSTEM

E.1. PARKING GARAGE STRUCTURE

E.1.1. INTRODUCTION

Layout
Along the Beam Length = 60 ft
Perpendicular to the Beam Length = 60 ft
Support Column Length = 24 in.
Corbel Length = 12 in.
Bearing Pad length = 6 in.
Clearance between End of Bearing and Edge of Corbel = 2 in.
Clearance between End of Beam and Edge of Column = 2 in.

E.1.2. MATERIALS

Cast-in-place concrete topping
- Structural thickness, \( t_s = 0 \) in.
- Specified concrete compressive strength for use in design, \( f_{c,\text{topping}} = 18.0 \) ksi
- Cast-in-place topping, \( E_c = 6,500 \) ksi
- Haunch thickness = 0 in.
- Haunch width = 0 in.
- Concrete unit weight = 0.150 kcf

Precast Concrete Beams
- Required concrete compressive strength at transfer, \( f_{ci} = 10.0 \) ksi
- Specified concrete compressive strength for use in design, \( f' = 18.0 \) ksi
- Required first-peak flexural strength at transfer, \( f_{1i} = 0.75 \) ksi
- Specified first-peak flexural strength for use in design, \( f_i = 1.0 \) ksi
- Specified peak flexural strength for use in design, \( f_p = 2.0 \) ksi
- Modulus of elasticity of concrete, \( E_c = 6,500 \) ksi
- Modulus of elasticity of concrete at transfer, \( E_{ci} = 5,000 \) ksi
- Concrete unit weight, \( w_c = 0.155 \) kcf
- Overall beam length, \( L = 57.83 \) ft
- Clear span (face of support to face of support), \( l_n = 56.33 \) ft

Prestressing strands: 0.6-in.-dia., seven-wire, low-relaxation
- Area of one strand = 0.217 in.\(^2\)
- Specified tensile strength, \( f_{pu} = 270.0 \) ksi
- Yield strength, \( f_{py} = 0.9f_{pu} = 243.0 \) ksi
- Stress limits for prestressing strands:
  - before transfer, \( f_p \leq 0.75f_{pu} = 202.5 \) ksi
  - at service limit state (after all losses), \( f_p \leq 0.80f_{py} = 194.4 \) ksi
- Modulus of elasticity, \( E_p = 28,500 \) ksi

Reinforcing bars:
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix E - Design Examples of Building Floor System

E.1.3. CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

E.1.3.1. Nontransformed Beam Section
Section properties over length of 19.28 ft @ the middle

- **bw** = web width = 2 in.
- **A_g** = area of cross section of beam = 498 in.$^2$
- **h** = overall depth of beam = 36.00 in.
- **I_g** = moment of inertia about the centroid of the noncomposite precast beam = 89,190 in.$^4$
- **y_b** = distance from centroid to extreme bottom fiber of the noncomposite precast beam = 13.89 in.
- **y_t** = distance from centroid to extreme top fiber of the noncomposite precast beam = 22.11 in.
- **S_b** = section modulus for the extreme bottom fiber of the noncomposite precast beam = 6,421 in.$^3$
- **S_t** = section modulus for the extreme top fiber of the noncomposite precast beam = 4,034 in.$^3$

Section properties at the end of the beam

- **bw** = web width = 5 in.
- **A_g** = area of cross section of beam = 654 in.$^2$
- **h** = overall depth of beam = 36.00 in.
- **I_g** = moment of inertia about the centroid of the noncomposite precast beam = 101,100 in.$^4$
- **y_b** = distance from centroid to extreme bottom fiber of the noncomposite precast beam = 15.11 in.
- **y_t** = distance from centroid to extreme top fiber of the noncomposite precast beam = 20.89 in.
- **S_b** = section modulus for the extreme bottom fiber of the noncomposite precast beam = 6,691 in.$^3$
- **S_t** = section modulus for the extreme top fiber of the noncomposite precast beam = 4,840 in.$^3$

- **w_{sw}** = average beam weight per unit length
  
  $w_{sw} = \frac{(498)(19.28) + 0.5(498 + 654)(2)(19.28)}{(57.83)(1/144)(0.155)} = 0.592 \text{ k/ft}$

E.1.3.2. Composite Section
Effective flange width is taken as the top flange width of the beam.

Effective Flange Width = 28 in.

Modular ratio between the topping and the beam concrete, $n = \frac{E_c(slab)}{E_c(beam)} = 1$

The effective flange width must be transformed by the modular ratio to provide cross-sectional properties equivalent to the girder concrete.

- Transformed flange width = $n$ (effective flange width) = 28.00 in.
- Transformed flange area = $n$ (effective flange width)($t_c$) = 0.00 in.$^2$
- Transformed flange moment of inertia = 0 in.$^4$

Consider the haunch in design?  No

- Transformed width of haunch = 0.00 in.
- Transformed area of haunch = 0.00 in.$^2$
- Transformed moment of inertia of haunch = 0 in.$^4$
### Table E.1.3.2-1. Properties of Composite Section at Mid Span

<table>
<thead>
<tr>
<th></th>
<th>Area in.²</th>
<th>$y_b$ in.</th>
<th>$A y_b$ in.³</th>
<th>$A(y_{bc} - y_b)$² in.⁴</th>
<th>$I$ in.⁴</th>
<th>$I + A(y_{bc} - y_b)^2$ in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>498</td>
<td>13.89</td>
<td>6,917</td>
<td>-</td>
<td>89,190</td>
<td>89,190</td>
</tr>
<tr>
<td>Haunch</td>
<td>0</td>
<td>36.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Deck</td>
<td>0</td>
<td>36.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Σ</td>
<td>498</td>
<td>-</td>
<td>6,917</td>
<td>-</td>
<td>-</td>
<td>89,190</td>
</tr>
</tbody>
</table>

- $A_c$ = total area of the composite section = 498 in.²
- $h_c$ = overall depth of the composite section = 36.00 in.
- $I_c$ = moment of inertia of the composite section = 89,190 in.⁴
- $y_{bc}$ = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam = 6,917/498 = 13.89 in.
- $y_{tg}$ = distance from the centroid of the composite section to the extreme top fiber of the precast beam = 36.00 − 13.89 = 22.11 in.
- $y_{tc}$ = distance from the centroid of the composite section to the extreme top fiber of the structural deck = 36.00 − 13.89 = 22.11 in.
- $S_{bc}$ = composite section modulus for the extreme bottom fiber of the precast beam
  $$S_{bc} = \left( \frac{I_c}{y_{bc}} \right) = \frac{89,190}{13.89} = 6,421 \text{ in.}^3$$
- $S_{tg}$ = composite section modulus for the extreme top fiber of the precast beam
  $$S_{tg} = \left( \frac{I_c}{y_{tg}} \right) = \frac{89,190}{22.11} = 4,034 \text{ in.}^3$$
- $S_{tc}$ = composite section modulus for extreme top fiber of the structural deck slab
  $$S_{tc} = \left( \frac{1}{n} \right) \left( \frac{I_c}{y_{tc}} \right) = \left( \frac{1}{1} \right) \left( \frac{89,190}{22.11} \right) = 4,034 \text{ in.}^3$$

### Table E.1.3.2-2. Properties of Composite Section at the End

<table>
<thead>
<tr>
<th></th>
<th>Area in.²</th>
<th>$y_b$ in.</th>
<th>$A y_b$ in.³</th>
<th>$A(y_{bc} - y_b)$² in.⁴</th>
<th>$I$ in.⁴</th>
<th>$I + A(y_{bc} - y_b)^2$ in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>654</td>
<td>15.11</td>
<td>9,881.94</td>
<td>-</td>
<td>101,100</td>
<td>101,100</td>
</tr>
<tr>
<td>Haunch</td>
<td>0</td>
<td>36.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Deck</td>
<td>0</td>
<td>36.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Σ</td>
<td>654</td>
<td>-</td>
<td>9,882</td>
<td>-</td>
<td>-</td>
<td>101,100</td>
</tr>
</tbody>
</table>

- $A_c$ = total area of the composite section = 654 in.²
- $h_c$ = overall depth of the composite section = 36.00 in.
- $I_c$ = moment of inertia of the composite section = 101,100 in.⁴
- $y_{bc}$ = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam = 9,882/654 = 15.11 in.
- $y_{tg}$ = distance from the centroid of the composite section to the extreme top fiber of the precast beam = 36.00 − 15.11 = 20.89 in.
- $y_{tc}$ = distance from the centroid of the composite section to the extreme top fiber of the structural deck = 36.00 − 15.11 = 20.89 in.
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix E - Design Examples of Building Floor System

\[ S_{bc} = \frac{I_c}{y_{bc}} = \frac{101,100}{15.11} = 6,691 \text{ in.}^3 \]

\[ S_{tg} = \frac{I_c}{y_{tg}} = \frac{101,100}{20.89} = 4,840 \text{ in.}^3 \]

\[ S_{tc} = \frac{1}{n} \frac{I_c}{y_{tc}} = \frac{1}{1} \left( \frac{101,100}{20.89} \right) = 4,840 \text{ in.}^3 \]

E.1.4. SHEAR FORCES AND BENDING MOMENTS

The self-weight of the beam acts on the simple-span structure, while the weight of DTs, superimposed dead loads (SIDL), and live loads act on the composite (if any), fixed-end(s) of the structure.

E.1.4.1. Dead Loads

DC = Dead load of structural components and nonstructural attachments

Dead loads acting on the simply supported beam:

\( w_{sw} = \text{Beam self-weight} = 0.592 \text{ kips/ft} \)

Dead loads placed on the fixed-end beam:

\( w_{DT} = \text{Weight of the 12DT30 = (78 psf)(57.67 ft) = 4.498 k/ft} \)

\( w_{Top} = \text{Weight of topping, if any = (0.15 pcf)(0 in./12)(60 ft) = 0 k/ft} \)

\( w_{SIDL} = \text{Superimposed dead loads = (5 psf)(60 ft) = 0.300 k/ft} \)

E.1.4.2. Live Loads

Design live load is assumed to be uniform distributed load of 0.04 kcf

\( w_{LL} = \text{Live load} = (0.04 \text{ kcf})(60 \text{ ft}) = 2.400 \text{ k/ft} \)

E.1.4.3. Unfactored Shear Forces and Bending Moments Due to Dead Load

For a simply supported beam with clear span \( l_n \) loaded with a uniformly distributed load \( w \), the shear force \( V_x \) and bending moment \( M_x \) at any distance \( x \) from the support are given by:

\[ V_x = w(0.5l_n - x) \quad \text{(Eq. E.1.4.1.2-1)} \]

\[ M_x = 0.5wx(l_n - x) \quad \text{(Eq. E.1.4.1.2-2)} \]

Using the above equations, values of shear forces and bending moments for a typical beam, under the self-weight of the beam, is shown in Table E.1.4.4-1.

For a fixed end beam with clear span \( l_n \) loaded with a uniformly distributed load \( w \), the shear force \( V_x \) and bending moment \( M_x \) at any distance \( x \) from the support are given by:

\[ V_x = \frac{(M_1 - M_2)}{l_n} + w(0.5l_n - x) \quad \text{(Eq. E.1.4.1.2-1)} \]

\[ M_x = \frac{(M_1 - M_2)}{x/l_n} + 0.5wx(l_n - x) - M_1 \quad \text{(Eq. E.1.4.1.2-2)} \]

where

\[ M_1 = \text{Fixed end moment at support 1} = \alpha_1 w l_n^2 \]

where

\[ \alpha_1 = 1/11 \text{ for a Fixed End} \]

\[ M_2 = \text{Fixed end moment at support 2} = \alpha_2 w l_n^2 \]

where

\[ \alpha_2 = 0 \text{ for simply supported} \]
α factors may be assumed per ACI 318-14 Art. 6.5. This provision provides a simplified method to calculate the bending moment due to the external forces. Secondary moments due to continues prestressing, if any, should be accounted for separately (ACI 318-14 Art. 5.3.11). However, these values are determined to limit the negative moment to not exceed the ultimate flexural capacity of the connection of each end presented in Section E.1.6 in this example.

Using the above equations, values of shear forces and bending moments for a typical beam, under weight of DT, topping, SIDL and LL are computed and shown in Table E.1.4.4-1.

### E.1.4.4 Unfactored Shear Forces and Bending Moments

**Table E.1.4.4-1. Unfactored Shear Forces and Bending Moments Due to Dead & Live Loads for a Typical Interior Beam**

<table>
<thead>
<tr>
<th>Distance x, ft</th>
<th>Section x/L</th>
<th>Beam Weight</th>
<th>DT Weight + Topping</th>
<th>SIDL</th>
<th>LL</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Shear Vd kips</td>
<td>Moment Md ft-kips</td>
<td>Shear Vdnc kips</td>
<td>Moment Mdnc ft-kips</td>
<td>Shear Vb kips</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>16.7</td>
<td>0.0</td>
<td>149.7</td>
<td>-15,570.0</td>
<td>9.9</td>
</tr>
<tr>
<td>0.25</td>
<td>0.004</td>
<td>16.5</td>
<td>49.80</td>
<td>148.6</td>
<td>-15,123.0</td>
<td>9.9</td>
</tr>
<tr>
<td>1.50</td>
<td>0.026</td>
<td>15.8</td>
<td>292.13</td>
<td>142.9</td>
<td>-12,935.0</td>
<td>9.5</td>
</tr>
<tr>
<td>5.63</td>
<td>0.10</td>
<td>13.3</td>
<td>1,013.89</td>
<td>124.4</td>
<td>-6,311.0</td>
<td>8.3</td>
</tr>
<tr>
<td>11.27</td>
<td>0.20</td>
<td>10.0</td>
<td>1,803.80</td>
<td>99.0</td>
<td>1,250.0</td>
<td>6.6</td>
</tr>
<tr>
<td>16.90</td>
<td>0.30</td>
<td>6.7</td>
<td>2,366.94</td>
<td>73.7</td>
<td>7,085.0</td>
<td>4.9</td>
</tr>
<tr>
<td>22.53</td>
<td>0.40</td>
<td>3.30</td>
<td>2,704.90</td>
<td>48.3</td>
<td>11,209.0</td>
<td>3.2</td>
</tr>
<tr>
<td>28.17</td>
<td>0.50</td>
<td>0.0</td>
<td>2,817.69</td>
<td>23.0</td>
<td>13,625.0</td>
<td>1.5</td>
</tr>
<tr>
<td>33.80</td>
<td>0.60</td>
<td>-3.3</td>
<td>2,704.90</td>
<td>-2.3</td>
<td>14,325.0</td>
<td>-0.2</td>
</tr>
<tr>
<td>39.43</td>
<td>0.70</td>
<td>-6.7</td>
<td>2,366.94</td>
<td>-27.7</td>
<td>13,313.0</td>
<td>-1.9</td>
</tr>
<tr>
<td>45.06</td>
<td>0.80</td>
<td>-10.0</td>
<td>1,803.80</td>
<td>-53.0</td>
<td>10,590.0</td>
<td>-3.6</td>
</tr>
<tr>
<td>50.70</td>
<td>0.90</td>
<td>-13.3</td>
<td>1,013.89</td>
<td>-78.4</td>
<td>6,147.0</td>
<td>-5.3</td>
</tr>
<tr>
<td>54.83</td>
<td>0.948</td>
<td>-15.8</td>
<td>292.13</td>
<td>-96.9</td>
<td>1,805.0</td>
<td>-6.5</td>
</tr>
<tr>
<td>56.08</td>
<td>0.970</td>
<td>-16.5</td>
<td>49.80</td>
<td>-102.6</td>
<td>309.0</td>
<td>-6.9</td>
</tr>
<tr>
<td>56.33</td>
<td>1.00</td>
<td>-16.7</td>
<td>0.00</td>
<td>-103.7</td>
<td>0.0</td>
<td>-6.9</td>
</tr>
</tbody>
</table>

### E.1.4.5 Load Combinations

Total factored load is taken as follows:

- **for Service Limit State:** check stresses in prestressed concrete components:
  \[ Q = 1.0D + 1.0L \]

- **for Strength Limit State:** check ultimate strength:
  Greater of:
  \[ Q = 1.4D \]
  \[ Q = 1.2D + 1.6L \]

[PCI DHB Eq. 4-30]
[PCI DHB Eq. 4-31]
Table E.1.4.5-1. Factored Shear Force and Bending Moments Due to Live and Dead Loads for a Typical Interior Beam

<table>
<thead>
<tr>
<th>Distance x, ft</th>
<th>Section x/l</th>
<th>Service Loads</th>
<th>Ultimate Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Shear V&lt;sub&gt;S&lt;/sub&gt;_SRV kips</td>
<td>Moment M&lt;sub&gt;S&lt;/sub&gt;_SRV kip-in.</td>
</tr>
<tr>
<td>0.00</td>
<td>0</td>
<td>256.2</td>
<td>-24,916</td>
</tr>
<tr>
<td>0.25</td>
<td>0.004</td>
<td>254.3</td>
<td>-24,150</td>
</tr>
<tr>
<td>1.50</td>
<td>0.026</td>
<td>244.5</td>
<td>-20,408</td>
</tr>
<tr>
<td>5.63</td>
<td>0.10</td>
<td>212.4</td>
<td>-9,085</td>
</tr>
<tr>
<td>11.27</td>
<td>0.20</td>
<td>168.4</td>
<td>3,805</td>
</tr>
<tr>
<td>16.90</td>
<td>0.30</td>
<td>124.6</td>
<td>13,705</td>
</tr>
<tr>
<td>22.53</td>
<td>0.40</td>
<td>80.6</td>
<td>20,643</td>
</tr>
<tr>
<td>28.17</td>
<td>0.50</td>
<td>36.8</td>
<td>24,622</td>
</tr>
<tr>
<td>33.80</td>
<td>0.60</td>
<td>-7.0</td>
<td>25,629</td>
</tr>
<tr>
<td>39.43</td>
<td>0.70</td>
<td>-51.0</td>
<td>23,671</td>
</tr>
<tr>
<td>45.06</td>
<td>0.80</td>
<td>-94.8</td>
<td>18,752</td>
</tr>
<tr>
<td>50.70</td>
<td>0.90</td>
<td>-138.8</td>
<td>10,851</td>
</tr>
<tr>
<td>54.83</td>
<td>0.95</td>
<td>-170.9</td>
<td>3,180</td>
</tr>
<tr>
<td>56.08</td>
<td>0.97</td>
<td>-180.7</td>
<td>545</td>
</tr>
<tr>
<td>56.33</td>
<td>1.00</td>
<td>-182.6</td>
<td>0</td>
</tr>
</tbody>
</table>

E.1.5. SERVICE LOAD STRESSES AT CRITICAL LOCATIONS
The required number of strands is usually governed by concrete tensile stresses at the bottom fiber for service load combination at the section of maximum moment.

E.1.5.1. Strand Pattern

The distance between the center of gravity of the strands and the bottom concrete fiber of the beam at midspan is:

\[ y_{bs} = \frac{18(2) + 12(4) + 4(34)}{34} = 6.47 \text{ in.} \]
e = yb - ya = 13.89 - 6.47 = 7.42 in. @ midspan

E.1.5.2. **Stress Calculations**

Stress due to applied dead and live loads using service load combination:

\[
S_{wc} = \frac{P_o}{A_g} + \frac{P_e \varepsilon}{S_{nc}} + \frac{M_d}{S_{nc}} + \frac{M_{dnc}}{S_{tc}} + \frac{M_{sd}}{S_{tc}} + \frac{M_t}{S_{tc}}
\]

where

- Assumed initial losses = 0% of \( P_t \)
- Assumed total losses = 30% of \( P_t \)

\[
P_j = \text{jacking force before losses} = 0.75 \rho_{f} P_o = (0.75)(34)(0.217)(270) = 1,494 \text{ kips}
\]

\[
P_o = \text{initial prestressing force after initial losses} = (1)(1,494) = 1,494 \text{ kips}
\]

\[
P_e = \text{effective prestressing force after all losses} = (0.7)(1,494) = 1,046 \text{ kips}
\]

\[
M_d = \text{unfactored bending moment due to beam self-weight, kip-in.}
\]

\[
M_{dnc} = \text{unfactored bending moment due to DT and topping weights, kip-in.}
\]

\[
M_{sd} = \text{unfactored bending moment due to superimposed dead load, kip-in.}
\]

\[
M_t = \text{unfactored bending moment due to live load, kip-in.}
\]

Using values of bending moments from **Table E.1.4.4-1**, the stresses at end and midspan points are:

**Table E.1.5.2-1. Calculation of Critical Stresses**

<table>
<thead>
<tr>
<th>Load</th>
<th>End Point at release ( P = P_o )</th>
<th>Midspan Point at release ( P = P_o )</th>
<th>Midspan Point at service ( P = P_o )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( f_0 )</td>
<td>( f_t )</td>
<td>( f_s )</td>
</tr>
<tr>
<td>( P/A_g )</td>
<td>2,284</td>
<td>2,284</td>
<td>3,000</td>
</tr>
<tr>
<td>( P_e/S_{nc} )</td>
<td>1,929</td>
<td>-2,667</td>
<td>1,726</td>
</tr>
<tr>
<td>( M_d/S_{nc} )</td>
<td>0</td>
<td>0</td>
<td>-421</td>
</tr>
<tr>
<td>( M_{dnc}/S_{tc} )</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( M_{sd}/S_{tc} )</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>( M_{t}/S_{tc} )</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Stresses</td>
<td>4,213</td>
<td>-383</td>
<td>4,305</td>
</tr>
<tr>
<td>Limiting Stress</td>
<td>0.70 ( f_o )’ ( f_i )</td>
<td>0.70 ( f_o )’ ( f_i )</td>
<td>0.70 ( f_o )’ ( f_i )</td>
</tr>
<tr>
<td></td>
<td>7,000</td>
<td>-750</td>
<td>7,000</td>
</tr>
<tr>
<td></td>
<td>OK</td>
<td>OK</td>
<td>OK</td>
</tr>
</tbody>
</table>

*Conservatively, the stresses were checked at the end of the beam at release, assuming the transfer length equals zero.*

**E.1.6. FLEXURAL STRENGTH OF THE MIDSPAN SECTION**

Ultimate bending moment based on **Table E.1.4.5-1** should be taken as the max of the following load combinations:

\[
M_o = \text{Max}[1.4(DL), 1.2(DL) + 1.6(LL)]
\]

\[
M_u = 1.2(2,704.90 + 14,325.0 + 955.54) + 1.6(7,643.33) = 2,817.67 \text{ kip-ft}
\]
In order to maintain equilibrium, the sum of the tension and compression forces must equal zero. The process is iterative by assuming the distance from the extreme compression fiber to the neutral axis \( c \). Construct a strain diagram as shown below and use strain compatibility to find \( c \).

Note that the iteration has already been done, and only the found value will be shown and used in this section.

\[ c = 7.52 \text{ in.} \]

\[ a = \text{depth of equivalent rectangular stress block} = \beta_1 c \]

where

\[ \beta_1 = \text{stress factor of compression block} \]

\[ = 0.85 \text{ for } f_c' \leq 4.0 \text{ ksi} \]

\[ = 0.85 - 0.05(f_c' - 4.0) \geq 0.65 \text{ for } f_c' > 4.0 \text{ ksi} \]

Therefore,

\[ \beta_1 = 0.85 - 0.05(18 - 4.0) \geq 0.65 \]

\[ = 0.15 < 0.65, \text{ use 0.65} \]

\[ a = (0.65)(7.52) = 4.88 \text{ in.} > \text{Top flange thickness} = 4 \text{ in.} \]

Therefore, the rectangular section behavior assumption is not valid.

The bottom strand layers may be represented by a group with the same stress-strain properties and effective prestress. This group has a combined area with a single centroid \( d_p \). The strain in the group is:

\[ \varepsilon_s = \varepsilon_c \left( \frac{d_p}{c} - 1 \right) + \left( \frac{f_{pe}}{E_p} \right) \]

where

\[ \varepsilon_c = \text{ultimate compressive strain} = 0.003 \text{ in./in.} \]

\[ d_p = \text{distance from extreme compression fiber to the centroid of the prestressing strands} \]

\[ = h + \text{Haunch Thickness} + t_s - y_{bs} \]

\[ y_{bs} = \text{distance between the center of gravity of bottom strands and the bottom concrete fiber} \]

\[ = [(18)(2) + (12)(4)]/30 = 2.8 \text{ in.} \]

\[ f_{pe} = \text{effective prestress} \]

\[ = (1 - 0.3)(0.75)(270) = 141.77 \text{ ksi} \]

Therefore,

\[ d_p = 36 + 0 + 0 - 2.8 = 33.2 \text{ in.} \]

\[ \varepsilon_s = 0.003 \left( \frac{33.2}{7.52} - 1 \right) + \left( \frac{141.77}{28,500} \right) = 0.015 \]

By proportional triangles from the strain diagram, the strain in the top strand layer is:

\[ \varepsilon' = \varepsilon_c \left( \frac{d'}{c} - 1 \right) + \left( \frac{f_{pe}}{E_p} \right) \]

where

\[ d' = \text{distance between the center of gravity of top strands and the top concrete fiber} = 2 \text{ in.} \]
Therefore,
\[ \varepsilon' = 0.003 \left( \frac{2}{7.52} - 1 \right) + \left( \frac{141.77}{28,500} \right) = 0.003 \]

Check the equilibrium of forces:
\[ \Sigma F = -C_1 - C_2 - C_3 - C_4 + T'_{ps} + T_{ps} \]
\[ \Sigma F = -0.85f'_c a_1 b_1 - 0.85f'_c a_2 b_2 + A'_{ps} (f'_{ps} + 0.85f'_c) + A_{ps} f_{ps} \]

where
\[ f'_c = \text{specified compressive strength of compression block} = 18.0 \text{ ksi} \]
\[ a_1 = \text{depth of equivalent rectangular stress block within the first layer} = 4 \text{ in.} \]
\[ b_1 = \text{equivalent width of the compression face of the member within the first layer} = 28 \text{ in.} \]
\[ a_2 = \text{depth of equivalent rectangular stress block within the second layer} \]
\[ b_2 = \text{equivalent width of the compression face of the member within the second layer} = 7 \text{ in.} \]
\[ f'_{ps} f_{ps} = \text{average stress of the prestressing strand in the top and bottom, respectively, ksi} \]
\[ = 28,500 \varepsilon_s \quad \text{For} \ \varepsilon_s \leq 0.0085 \]  
\[ = 270 - \frac{0.04}{\varepsilon_s - 0.007} \quad \text{For} \ \varepsilon_s \geq 0.0085 \]

\[ A'_{ps} = \text{area of top prestressing strands} = (4)(0.217) = 0.87 \text{ in.}^2 \]
\[ A_{ps} = \text{area of bottom prestressing strands} = (30)(0.217) = 6.51 \text{ in.}^2 \]

Therefore,
\[ f_{ps} = 270 - 0.04/(0.015 - 0.007) = 265.14 \text{ ksi} \]
\[ f'_{ps} = (0.003)(28,500) = 79.03 \text{ ksi} \]
\[ C_1 = 0.85(18)(4)(28) = 1,713.6 \text{ kips} \]
\[ C_2 = 0.85(18)(0.88)(7) = 94.2 \text{ kips} \]
\[ T'_{ps} = 0.87[79.03 + 0.85(18)] = 81.9 \text{ kips} \]
\[ T_{ps} = (6.51)(265.14) = 1,726.1 \text{ kips} \]

Sum of compression and tension forces:
\[ \Sigma F = -0.85(18)(4)(28) - 0.85(18)(0.88)(7) - 0.85(18)(0)(4) - 0.85(18)(0)(7) + 0.868[79.03 + 0.85(18)] + 6.51(265.14) \]
\[ \Sigma F = -1,713.6 - 94.2 - 0 - 81.9 + 1,726.1 = 0 \]

Nominal flexural resistance:
\[ M_n = C_1 (c - \frac{a_1}{2}) + C_2 (c - a_1 - \frac{a_2}{2}) + C_3 (c - a_1 - a_2 - \frac{a_3}{2}) - T'_{ps} (c - d') + T_{ps} (d_p - c) \]
\[ M_n = 1,713.6 \left( 7.52 - \frac{4}{2} \right) + 94.2 \left( 7.52 - 4 - \frac{0.88}{2} \right) - 81.9(7.52 - 2) + 1,726.1(33.2 - 7.52) \]
\[ M_n = 53,623.37 \text{ kip-in.} = 4,468.61 \text{ ft-kip} \]

Reduced flexural resistance:
\[ M_r = \phi M_n \]

where
\[ \phi = \text{resistance factor} \]
\[ = 0.65 \leq \phi = 0.65 + \frac{0.25(\varepsilon_t - \varepsilon_{ty})}{(0.005 - \varepsilon_{ty})} \leq 0.90 \]

[ACI 318-14 21.2.2]
where
\[ \varepsilon_t = \varepsilon_s = 0.015 \]
\[ \varepsilon_y = \frac{f_y}{E_s} = 60/29,000 = 0.002 \]
Therefore,
\[ \phi = 0.65 + \frac{0.25(0.015 - 0.002)}{0.005 - 0.002} = 1.77 \]
\[ = 1.77 > 0.90, \text{ use } \phi = 0.90 \]
\[ M_r = (0.90)(4,468.61) = 4,022 \text{ ft-kips} > M_u = 2,818 \text{ ft-kips} \quad \text{OK} \]

E.1.7. **FLEXURAL STRENGTH AT THE FIXED END**

E.1.8. **SHEAR STRENGTH AT THE CRITICAL SECTION NEAR THE SIMPLE SUPPORT**

The area and spacing of shear reinforcement, if any, must be determined at regular intervals along the entire length of the beam. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports.

Transverse shear reinforcement is required when:

\[ V_u > \phi(V_c + V_f + V_p) \]

where
\[ V_u = \text{total factored shear force, kips} \]
\[ V_c = \text{nominal shear resistance provided by tensile stresses in the concrete, kips} \]
\[ V_f = \text{nominal shear resistance provided by tensile stresses in the fibers, kips} \]
\[ V_p = \text{component in the direction of the applied shear of the effective prestressing force, kips} \]
\[ \phi = \text{resistance factor} = 0.75 \]

E.1.8.1. **Critical Section**

The critical section near the supports is taken as the effective shear depth, \( d_v \), from the internal face of the support.

\[ d_v = \text{the distance between the resultants of the tensile and compressive forces, } (d_c - a/2), \text{ but not less than } (0.9d_e) \text{ or } (0.72h_c) \]

where
\[ d_e = \text{the corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement} \]
\[ a = \text{depth of compression block (at midspan, assumed adequate) = 4.88 in.} \]
\[ h_c = \text{overall depth of the composite section = 36 in.} \]

The effective depth, \( d_v \), is calculated based on the center of gravity of the bottom strands at the end of the critical section

\[ d_e = h_c \cdot y_{bs} = 36.00 \cdot 2.80 = 33.20 \text{ in.} \]
\[ d_v = 13.20 - (4.88/2) = 30.76 \text{ in.} \]
\[ > 0.9d_e = 0.9(33.20) = 29.88 \text{ in.} \]
\[ > 0.72h_c = 0.72(36.00) = 25.92 \text{ in.} \]

Therefore, \( d_v = 30.76 \text{ in.} \)

Because the width of the bearing is not yet determined, it is conservatively assumed to be zero. Therefore, the critical section in shear is located at a distance of \( h_c/2 \) from the end of the beam:

\[ h_c/2 = 36/12/2 = 1.5 \text{ ft} \]
**E.1.8.2. Contribution of Concrete to Nominal Shear Resistance**

The contribution of the concrete to the nominal shear resistance is:

\[ V_c = 0.0316 \beta \sqrt{f'_t b_v d_v} \]  

where

\[ \beta = \text{a factor indicating the ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution).} \]

Several quantities must be determined before this expression can be evaluated.

**E.1.8.2.1. Strain in Flexural Tension Reinforcement**

Calculate the strain at the centroid of the tension reinforcement, \( \varepsilon_s \):

\[
\varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0.5N_u + |(V_u - V_p)| - A_{ps} f_{po}}{(E_s A_s + E_p A_{ps})} 
\]

\[ \text{[LRFD Eq. 5.7.3.4.2-4]} \]

where

- \( M_u \) = applied factored bending moment at the critical section = 4,201 kip-in.
- \( N_u \) = applied factored axial force at the critical section = 0 kips
- \( V_u \) = applied factored shear force at the critical section = 166.9 kips
- \( V_p \) = (Force per strand without live load gains)(Number of harped strands)(sin \( \psi \)) = 0
- \( A_{ps} \) = area of prestressing strands on the flexural tension side of the member = 30(0.217) = 6.51 in.\(^2\)
- \( f_{po} \) = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi).

For pretensioned members, LRFD Article 5.7.3.4.2 indicates that \( f_{po} \) can be taken as \( 0.7 f_{pu} \). (Note: use this for both pretensioned and post-tensioned systems made with stress relieved and low relaxation strands).

\[ 0.7(270) = 189.0 \text{ ksi} \]

Therefore,

\[
\varepsilon_s = \frac{\frac{4,201}{30.76} + 0.05(0) + |(166.9 - 0)| - (6.51)(189)}{(0 + 28,500(6.51))} = -0.005
\]

\( \varepsilon_s \) is less than zero. Use \( \varepsilon_s = 0 \)

**E.1.8.2.2. Values of \( \beta \) and \( \theta \)**

Assume the section contains at least the minimum amount of transverse reinforcement:

\[ \beta = \frac{4.8}{1 + 750 \varepsilon_s} = \frac{4.8}{1 + 750(0)} = 4.8 \]  

\[ \text{[LRFD Eq. 5.7.3.4.2-1]} \]

Angle of diagonal compressive stresses is:

\[ \theta = 29 + 3,500 \varepsilon_s = 29 + 3,500(0) = 29.0^\circ \]  

\[ \text{[LRFD Eq. 5.7.3.4.2-3]} \]

**E.1.8.2.3. Compute Concrete Contribution**

The nominal shear resisted by the concrete is:

\[ V_c = 0.0316 \beta \sqrt{f'_t b_v d_v} \]  

\[ \text{[LRFD Eq. 5.7.3.3-3]} \]

where \( b_v \) = effective web width = 9.5 in.

\[ V_c = 0.0316(4.8) \sqrt{180(9.5)(30.76)} = 188.1 \text{ kips} \]

**E.1.8.3. Contribution of Fiber to Nominal Shear Resistance**

The nominal shear resisted by the fiber is:

\[ V_f = \psi f_{fu} \cot \theta b_v d_v \]
where
\( \psi f_u = \) residual rupture stress = 0.75 ksi

Therefore,
\[ V_f = (0.75) \cot(29.0^\circ)(9.5)(30.76) = 395.4 \text{ kips} \]

### E.1.8.4. Contribution to Transverse Reinforcement to Nominal Shear Resistance

Check if \( V_u > \Phi (V_c + V_f + V_p) \)

\[ \Phi (V_c + V_f + V_p) = (0.75)(188.1 + 395.4 + 0) = 437.6 \text{ kips} > 166.9 \text{ kips} \quad \text{OK} \]

Therefore, transverse shear reinforcement is not needed.

### E.1.8.5. Maximum Nominal Shear Resistance

In order to ensure that the concrete in the web of the beam will not crush prior to pull out of the fibers, AFGC (2013) gives an upper limit of \( V_n \) as the ultimate strength of the compressive struts as follows:

\[ V_n = 0.18 f'_c b_v d_v + V_p \quad [\text{LRFD 5.7.3.3-2}] \]

Comparing this equation with LRFD Eq. 5.8.3.3-1, it can be concluded that:

\( V_c + V_s + V_f \) must not be greater than \( V_n = 0.18 f'_c b_v d_v \)

\( V_c + V_s + V_f = 260.5 + 0 + 547.8 = 808.3 \text{ kips} \)

\( V_n = 0.18 f'_c b_v d_v = 0.18(18.0)(9.5)(30.76) = 946.8 \text{ kips} \)

\( V_n = 946.8 \text{ kips} > V_c + V_s + V_f = 808.3 \text{ kips} \quad \text{OK} \)

Using the above procedures, the transverse reinforcement can be determined at increments along the entire length of the beam.

### E.1.9. Shear Strength at the Critical Section Near the Fixed End
F. APPENDIX F - DESIGN EXAMPLES OF BRIDGE BEAMS

F.1. Transformed Sections, Shear General Procedure, Refined Losses

F.1.1. INTRODUCTION
This design example demonstrates the design of a 250-ft, single span, UHPC Decked-I beam bridge with no skew. This example illustrates in detail the design of a typical interior beam at the critical sections in positive flexure, shear, and deflection due to prestress, dead loads, and live load. The superstructure consists of five beams spaced at 9 ft 0 in. centers, as shown in Figure F.1.1-1. In order to optimize the proposed 9 ft wide top flange, it is proposed to use a ribbed slab system similar to that proposed by FHWA and implemented by Iowa DOT, as a stand-alone precast deck panel system. The general slab depth is 2 in., which is ribbed at 2 ft 0 in. with 8-in.-deep ribs. A ¼-in.-thick surface preparation is considered to be an integral part of the deck. Design live load is HL-93. The design is accomplished in accordance with the AASHTO LRFD Bridge Design Specifications, Eighth Edition, 2016. Elastic stresses from external loads are calculated using transformed sections. Shear strength is calculated using the modified compression theory. Time-dependent prestress losses are calculated using the refined estimates.

![Figure F.1.1-1. Bridge Cross Section](image)

- Number of Girders, N_h = 5
- Girder Spacing = 9.00 ft
- Overhang = 4.33 ft
- Max. Barrier Width = 1.50 ft
- Edge-to-edge width of bridge = 44.67 ft
- Number of Design Lanes = 3
- Skew Angle = 0°

The following terminology is used to describe cross sections in this design example:
- Nontransformed section—the concrete beam cross section without the strands transformed. Also called the gross section.
- Transformed section—the concrete beam plus the strands transformed to provide cross-sectional properties equivalent to the girder concrete.

The term "transformed" generally refers to transformation of the strands.

F.1.2. MATERIALS
Cast-in-place concrete slab: Actual thickness = 0.00 in.
Structural thickness = 0.00 in.
Note that a ½-in.-thick wearing surface is considered an integral part of the deck thickness.

Specified concrete compressive strength for use in design, \( f'_{c,\text{deck}} = 4.0 \text{ ksi} \)

Modulus of elasticity of cast-in-place slab, \( E_{c,\text{deck}} = 3,834 \text{ ksi} \)

Haunch Thickness = 0.00 in.
Haunch Width = 108.00 in.
Concrete unit weight = 0.150 kcf

Precast Concrete Beams

Required concrete compressive strength at transfer, \( f'_{c}=10.0 \text{ ksi} \)
Specified concrete compressive strength for use in design, \( f'_{c} = 18.0 \text{ ksi} \)
Required first-peak tensile strength at transfer, \( f_{1i} = 0.75 \text{ ksi} \)
Specified first-peak tensile strength for use in design, \( f_{1} = 1.0 \text{ ksi} \)
Specified peak flexural strength for use in design, \( f_{p} = 2.0 \text{ ksi} \)
Modulus of elasticity of concrete, \( E_{c} = 6,500 \text{ ksi} \)
Modulus of elasticity of concrete at transfer, \( E_{c,\text{ci}} = 5,000 \text{ ksi} \)
Concrete unit weight, \( w_{c} = 0.155 \text{ kcf} \)
Overall beam length, \( L = 252 \text{ ft} \)
Design span, \( l_{CL-CL} = 250 \text{ ft} \)

Prestressing strands: 0.7-in.-dia., seven-wire, low-relaxation

Area of one strand = 0.294 in.\(^2\)
Specified tensile strength, \( f_{pu} = 270.0 \text{ ksi} \)
Yield strength, \( f_{py} = 0.9f_{pu} = 243.0 \text{ ksi} \) [LRFD Table 5.4.4.1-1]
Stress limits for prestressing strands:
  • before transfer, \( f_{ps} \leq 0.75f_{pu} = 202.5 \text{ ksi} \) [LRFD Table 5.9.3-1]
  • at service limit state (after all losses), \( f_{ps} \leq 0.80f_{py} = 194.4 \text{ ksi} \)
Modulus of elasticity, \( E_{p} = 28,500 \text{ ksi} \) [LRFD Art. 5.4.4.2]

Reinforcing bars:
Yield strength, \( f_{y} = 60.0 \text{ ksi} \)
Modulus of elasticity, \( E_{s} = 29,000 \text{ ksi} \)

Future wearing surface: 2 in. additional concrete, unit weight = 0.150 kcf with thickness of 2

New Jersey-type barrier: unit weight = 0.300 kips/ft/side

F.1.3. CROSS-SECTION PROPERTIES FOR A TYPICAL INTERIOR BEAM

F.1.3.1. Nontransformed Beam Section
The following properties are calculated as the weighted average of two sections at the rib and at the top skin. (It also should be noted that these properties for the cross section with top flange of 8'-8" wide as 07/09/20F. The top flange was changed to 8'-6". The calculations will be revised.)
Appendix F - Design Examples of Bridge Beams

\begin{align*}
\text{Area of cross section of beam} & = 1127 \text{ in.}^2 \\
\text{Overall depth of beam} & = 108.00 \text{ in.} \\
\text{Moment of inertia about the centroid of the noncomposite precast beam} & = 2,102,010 \text{ in.}^4 \\
\text{Distance from centroid to extreme bottom fiber of the noncomposite precast beam} & = 58.34 \text{ in.} \\
\text{Distance from centroid to extreme top fiber of the noncomposite precast beam} & = 49.66 \text{ in.} \\
\text{Section modulus for the extreme bottom fiber of the noncomposite precast beam} & = 36,030 \text{ in.}^3 \\
\text{Section modulus for the extreme top fiber of the noncomposite precast beam} & = 42,328 \text{ in.}^3 \\
\text{St. Venant's torsional inertia} & = 18,389 \text{ in.}^4 \\
\text{Volume-to-surface ratio of the beam} & = 3.00 \\
\text{Beam weight per unit length} & = 1.210 \text{ kips/ft} \\
\text{Modulus of elasticity of concrete} & = 33,000K_1(w_c)^{1.5}\sqrt{f'_c} \quad \text{[LRFD Eq. 5.4.2.4-1]}
\end{align*}

**Figure F.1.3.1-1. Dimensions of the Composite Section**

**F.1.3.2. Composite Section**

**F.1.3.2.1. Effective Flange Width**

Effective flange width is taken as the tributary width perpendicular to the axis of the beam. For the interior beam, the effective flange width is calculated as one-half the distance to the adjacent beam on each side.

Effective Flange Width = 108.00 in.

**F.1.3.2.2. Modular Ratio between Slab and Beam Concrete**

Modular ratio between slab and beam concrete, \( n = \frac{E_c(\text{slab})}{E_c(\text{beam})} = 0.5898 \)

**F.1.3.2.3. Section Properties**

The effective flange width must be transformed by the modular ratio to provide cross-sectional properties equivalent to the girder concrete. It should be noted the CIP closure joint is ignored in the following calculations.

Transformed flange width = \( n(\text{effective flange width}) = (0.5898)(108) = 63.70 \text{ in.} \)

Transformed flange area = \( n(\text{effective flange width})(t_s) = 0.00 \text{ in.}^2 \)
Transformed flange moment of inertia = 0 in.\(^4\)

Consider the haunch in design?  No

Transformed width of haunch = 0.00 in.

Transformed area of haunch = 0.00 in.\(^2\)

Transformed moment of inertia of haunch = 0 in.\(^4\)

### Table F.1.3.2.3-1. Properties of Composite Section

<table>
<thead>
<tr>
<th></th>
<th>Area in.(^2)</th>
<th>(y_b) in.</th>
<th>(Ay_b) in.(^3)</th>
<th>(A(y_{bc} - y_b)) in.(^4)</th>
<th>(I) in.(^4)</th>
<th>(I + A(y_{bc} - y_b))(^2) in.(^4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>1,127</td>
<td>58.34</td>
<td>65,749</td>
<td>-</td>
<td>2,102,000</td>
<td>2,102,010</td>
</tr>
<tr>
<td>Haunch</td>
<td>0</td>
<td>108.00</td>
<td>-</td>
<td>-</td>
<td>0.000</td>
<td>-</td>
</tr>
<tr>
<td>Deck</td>
<td>0</td>
<td>108.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(\Sigma)</td>
<td>1,127</td>
<td>-</td>
<td>65,749</td>
<td>-</td>
<td>-</td>
<td>2,102,010</td>
</tr>
</tbody>
</table>

\(A_c\) = total area of the composite section = 1,127 in.\(^2\)

\(h_c\) = overall depth of the composite section = 108.00 in.

\(I_c\) = moment of inertia of the composite section = 2,102,010 in.\(^4\)

\(y_{bc}\) = distance from the centroid of the composite section to the extreme bottom fiber of the precast beam = 65,749/1,127 = 58.34 in.

\(y_{tg}\) = distance from the centroid of the composite section to the extreme top fiber of the precast beam = 108.00 – 58.34 = 49.66 in.

\(y_{tc}\) = distance from the centroid of the composite section to the extreme top fiber of the structural deck = 108.00 – 58.34 = 49.66 in.

\(S_{bc}\) = composite section modulus for the extreme bottom fiber of the precast beam

\[ (\frac{I_c}{y_{bc}}) = \frac{2,102,010}{58.34} = 36,030 \text{ in.}^3 \]

\(S_{tg}\) = composite section modulus for the extreme top fiber of the precast beam

\[ (\frac{I_c}{y_{tg}}) = \frac{2,102,010}{58.34} = 42,328 \text{ in.}^3 \]

\(S_{tc}\) = composite section modulus for extreme top fiber of the structural deck slab

\[ (\frac{1}{n})(\frac{I_c}{y_{tc}}) = (\frac{1}{0.5898})(\frac{2,102,010}{49.66}) = 71,767 \text{ in.}^3 \]

**F.1.4.  SHEAR FORCES AND BENDING MOMENTS**

The self-weight of the beam and the weight of the deck and haunch act on the noncomposite, simple-span structure, while the weight of barriers, future wearing surface, and live loads with impact act on the composite, simple-span structure.
F.1.4.1. Shear Forces and Bending Moments Due to Dead Loads

F.1.4.1.1. Dead Loads

\( \text{DC} = \text{Dead load of structural components and nonstructural attachments} \)

Dead loads acting on the noncomposite structure:

- Beam self-weight, \( w_g = 1.210 \text{ kips/ft} \)
- Deck weight = \( \frac{(0/12)(108/12)(0.15)}{(108/12)(0.15)} = 0.000 \text{ kips/ft} \)
- Haunch weight = \( \frac{(0/12)(108/12)(0.15)}{(108/12)(0.15)} = 0.000 \text{ kips/ft} \)

Dead loads placed on the composite structure:

LRFD Article 4.6.2.2.1 states that permanent loads (curbs and wearing surface) may be distributed uniformly among all beams if the following conditions are met:

- Width of the deck is constant \( \text{OK} \)
- Number of beams, \( N_b \), is not less than four \( \text{OK} \)
- Beams are parallel and have the same stiffness \( \text{OK} \)
- The roadway part of the overhang, \( d_e \leq 3.0 \text{ ft.} \)

\( d_e = 4.33 - 1.50 = 2.83 \text{ ft.} \) \( \text{OK} \)
- Curvature in plan is less than specified in the LRFD Specifications \( \text{OK} \)
- Cross section of the bridge is consistent with one of the cross sections given in LRFD Table 4.6.2.1-1 \( \text{OK} \)

Since these criteria are satisfied, the barrier and wearing surface loads are distributed equally among the 5 beams.

- \( w_b = \text{barrier weight} = \frac{(2 \text{ barriers})(0.3 \text{ kips/ft})}{(5 \text{ beams})} = 0.120 \text{ kips/ft/beam} \)
- \( w_{ws} = \text{dead load of wearing surface} = \frac{(2/12)(0.15)(41.67 \text{ ft})}{(5 \text{ beams})} = 0.208 \text{ kips/ft/beam} \)

\[ V_x = w(0.5L - x) \]  
\[ M_x = 0.5wx(L - x) \]  
(Eq. F.1.4.1.2-1)  
(Eq. F.1.4.1.2-2)

Using the above equations, values of shear forces and bending moments for a typical interior beam, under self-weight of beam, weight of slab and haunch, weight of barriers and wearing surface are computed and shown in Table F.1.4-1 that is located at the end of Section F.1.4.3. For these calculations, the span length \( (L) \) is the design span, 250-ft. However, for calculations of stresses and deformation at the time prestress is transferred, the overall length of the precast member, 252-ft, is used as illustrated later in this example.

F.1.4.2. Shear Forces and Bending Moments Due to Live Loads

F.1.4.2.1. Live Loads

Design live load is HL-93, which consists of a combination of:

1. Design truck or design tandem with dynamic allowance \( \text{[LRFD Art. 3.6.1.2.1]} \)
   The design truck consists of 8.0-, 32.0-, and 32.0-kip axles with the first pair spaced at 14-ft and the second pair spaced at 14- to 30-ft. The design tandem consists of a pair of 25.0-kip axles spaced at 4-ft apart. \( \text{[LRFD Art. 3.6.1.2.2]} \)
2. Design lane load of 0.64 kips/ft without dynamic allowance \( \text{[LRFD Art. 3.6.1.2.4]} \)
F.1.4.2.2. Live Load Distribution Factors for a Typical Interior Beam

Cross Section Type

Is there concrete deck on the beams? NO

The live load bending moments are determined by using the simplified distribution factor formulas, [LRFD Art. 4.6.2.2]. To use the simplified live load distribution factor formulas, the following conditions must be met: [LRFD Art. 4.6.2.2.1].

- Width of the deck is constant OK
- Beams are parallel and have the same stiffness OK
- Curvature in plan is less than specified in the LRFD Specifications OK

F.1.4.2.2.1. Distribution Factor for Bending Moment

LRFD Table 4.6.2.2.2b-1 is applicable for all limit states except fatigue limit state.

Regardless of Number of Loaded Lanes:

\[ DFM = \frac{S}{D} \]  

[LRFD Table 4.6.2.2.2b-1]

Provided that: \( \text{Skew} \leq 45^\circ; \quad \text{Skew} = 0^\circ \quad \text{OK} \)

\[ 5 \leq N_L \leq 20 \quad N_L = 5 \quad \text{OK} \]

where

\[ DFM = \text{distribution factor for bending moment for interior beam} \]
\[ D = \text{width of distribution per lane, ft} \]
\[ = 11.5 - N_L \times 1.4N_L(1 - 0.2C)^2 \text{when } C \leq 5 \]  

[LRFD Table 4.6.2.2.2b-1]
\[ = 11.5 - N_L \text{ when } C > 5 \]  

[LRFD Table 4.6.2.2.2b-1]
\[ C = \text{stiffness parameter} \]
\[ = K(W/L) \leq K \]  

[LRFD Table 4.6.2.2.2b-1]

where

\[ W = \text{edge-to-edge width of bridge, ft} \]
\[ K = \text{constant for different types of construction} \]
\[ = \sqrt{\frac{(1 + \mu)I}{J}} \]
\[ \mu = \text{Poisson’s ratio} = 0.2 \]

Therefore,

\[ K = \sqrt{\frac{(1 + 0.2)(2269510)}{19741}} = 11.75 \]
\[ C = (1.81)(44.67)/250 = 0.32 \]
\[ D = 11.5 - 3 + 1.4(3)(1 - 0.2*0.32)^2 = 12.17 \]

\[ DFM = 9/12.17 = 0.74 \text{ lanes/beam} \]

- For Fatigue Limit State:

The LRFD Specifications, Art. C3.4.1, states that for Fatigue Limit State, a single design truck should be used. However, live load distribution factors given in LRFD Article 4.6.2.2 take into consideration the multiple presence
factor, \( m \). LRFD Article 3.6.1.2 states that the multiple presence factor, \( m \), for one design lane loaded is 1.2. Therefore, the distribution factor for one design lane loaded with the multiple presence factor removed, should be used. The distribution factor for the fatigue limit state is: \( 0.74/1.2 = 0.62 \) lanes/beam

### F.1.4.2.2.2. Distribution Factor for Shear Force

LRFD Table 4.6.2.2.3a-1 is applicable

The live load shear forces are determined by using the simplified distribution factor formulas, [LRFD Art. 4.6.2.2]. To use the simplified live load distribution factor formulas, the following conditions must be met: [LRFD Art. 4.6.2.2.3]

- Width of the deck is constant: OK
- Beams are parallel and have the same stiffness: OK
- Curvature in plan is less than specified in the LRFD Specifications: OK

Use the Lever Rule for one or more lanes loaded (see Section F.1.4.2.2.2.1).

![Diagram of Lever Rule](image)

**Figure F.1.4.2.2.2-1. Diagram of Lever Rule**

One Loaded Lane with Multiple Presence Factor: \( m = 1.2 \)

\[
DF = 1.2 \frac{(9 \text{ ft}) \frac{P}{2} + (3 \text{ ft}) \frac{P}{2}}{(9 \text{ ft})(P)} = 0.800 \text{ lanes/beam}
\]

Two Loaded Lane with Multiple Presence Factor: \( m = 1.0 \)

\[
DF = 1.0 \frac{(9 \text{ ft}) \frac{P}{2} + (3 \text{ ft}) \frac{P}{2} + (5 \text{ ft}) \frac{P}{2} + (0 \text{ ft}) \frac{P}{2}}{(9 \text{ ft})(P)} = 0.94 \text{ lanes/beam}
\]

### F.1.4.2.3. Dynamic Allowance

\( IM = 15\% \) for fatigue limit state

\( IM = 33\% \) for all other limit states

[LRFD Table 3.6.2.1-1]

where \( IM = \) dynamic load allowance, applied to design truck load only

### F.1.4.2.4. Unfactored Shear Forces and Bending Moments

#### F.1.4.2.4.1. Due to Truck Load; \( V_{LT} \) and \( M_{LT} \)

- For all limit states except for fatigue limit state:

Shear force and bending moment envelopes on a per-lane-basis are calculated at tenth-points of the span using the equations given in Chapter 8 of PCI Bridge Design Manual. However, this is generally done by means of commercially available computer software that has the ability to deal with moving loads. Therefore, truck load shear forces and bending moments per beam are:
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

\[ V_{LT} = (\text{shear force per lane})(DFV)(1 + IM) \]
\[ = (\text{shear force per lane})(0.944)*(1 + 0.33) \]
\[ = (\text{shear force per lane})(1.256) \text{ kips} \]

\[ M_{LT} = (\text{bending moment per lane})(DFM)(1 + IM) \]
\[ = (\text{bending moment per lane})(0.944)*(1 + 0.33) \]
\[ = (\text{bending moment per lane})(1.256) \text{ ft-kips} \]

Values of \( V_{LT} \) and \( M_{LT} \) at different points are given in Table F.1.4.2.

- For fatigue limit state:

Art. 3.6.1.4.1 in the LRFD Specifications states that the fatigue load is a single design truck which has the same axle weight used in all other limit states but with a constant spacing of 30-ft between the 32.0-kip axles. Bending moment envelope on a per-lane-basis is calculated using the equation given in Chapter 8 of PCI Bridge Design Manual.

\[ M_{LT} = (\text{bending moment per lane})(DFM)(1 + IM) \]
\[ = (\text{bending moment per lane})(0.616)*(1 + 0.15) \]
\[ = (\text{bending moment per lane})(0.708) \text{ ft-kips} \]

Values of \( M_{LT} \) at different points are given in Table F.1.4.2.

F.1.4.2.4.2. Due to Design Lane Load; \( V_{LL} \) and \( M_{LL} \)

To obtain the maximum shear force at a section located at a distance \( (x) \) from the left support under a uniformly distributed load of 0.64 kips/ft, load the member to the right of section under consideration as shown in Figure F.1.4.2.4.2-1.

\[ V_x = \frac{0.32(L - x)^2}{L} \]

Where \( V_x \) is in kips/lane and \( L \) and \( x \) are in ft.

![Figure F.1.4.2.4.2-1. Maximum Shear Force Due to Design Lane Load](image)

Figure F.1.4.2.4.2-1. Maximum Shear Force Due to Design Lane Load

To calculate the maximum bending moment at any section, use Eq. (F.1.4.1.2-2).

Lane load shear force and bending moment per typical interior beam are as follows:

\[ V_{LL} = (\text{lane load shear force})(DFV) \]
\[ = (\text{lane load shear force})(0.944) \text{ kips} \]

For all limit states except for fatigue limit state:

\[ M_{LL} = (\text{lane load bending moment})(DFM) \]
\[ = (\text{lane load bending moment})(0.944) \text{ ft-kips} \]
Note that the dynamic allowance is not applied to the design lane loading.

Values of shear forces and bending moments, \( V_{LL} \) and \( M_{LL} \), are given in **Table F.1.4-2.**

### F.1.4.3. Load Combinations

Total factored load is taken as:

\[
Q = \sum \eta_i \gamma_i Q_i
\]

where

- \( \eta_i \) = a load modifier relating to ductility, redundancy, and operational importance (Here, \( \eta_i \) is considered to be 1.0 for typical bridges)
- \( \gamma_i \) = load factors
- \( Q_i \) = force effects from specified loads

Investigating different limit states given in LRFD Article 3.4.1, the following limit states are applicable:

**Service I:** check compressive stresses in prestressed concrete components:

\[
Q = 1.00(DC + DW) + 1.00(LL + IM)
\]

This load combination is the general combination for service limit state stress checks and applies to all conditions other than Service III.

**Service III:** check tensile stresses in prestressed concrete components:

\[
Q = 1.00(DC + DW) + 0.80(LL + IM)
\]

This load combination is a special combination for service limit state stress checks and applies only to tension in prestressed concrete structures to control cracks.

**Strength I:** check ultimate strength:

- Maximum \( Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM) \)
- Minimum \( Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM) \)

This load combination is the general load combination for strength limit state design.

Note: For simple-span bridges, the maximum load factors produce maximum effects. However, use minimum load factors for dead load \( DC \), and wearing surface \( DW \) when dead load and wearing surface stresses are opposite to those of live load.

**Fatigue I:** check stress range in strands:

\[
Q = 1.50(LL + IM)
\]

This load combination is a special load combination to check the tensile stress range in the strands due to live load and dynamic allowance.
Table F.1.4-1. Unfactored Shear Forces and Bending Moments Due to Dead Loads for a Typical Interior Beam

<table>
<thead>
<tr>
<th>Distance x, ft</th>
<th>Section x/L</th>
<th>Beam Weight</th>
<th>Slab + Haunch Weight</th>
<th>Barrier Weight</th>
<th>Wearing Surface Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Beam Weight</td>
<td>Shear ( V_g ) kips</td>
<td>Moment ( M_g ) ft-kips</td>
<td>Shear ( V_s ) kips</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>151.3</td>
<td>0.0</td>
<td>0.0</td>
<td>15.0</td>
</tr>
<tr>
<td>8.53</td>
<td>0.034</td>
<td>141.0</td>
<td>1,232.6</td>
<td>0.0</td>
<td>14.0</td>
</tr>
<tr>
<td>25</td>
<td>0.1</td>
<td>121.0</td>
<td>3,403.1</td>
<td>0.0</td>
<td>12.0</td>
</tr>
<tr>
<td>50</td>
<td>0.2</td>
<td>90.8</td>
<td>6,050.0</td>
<td>0.0</td>
<td>9.0</td>
</tr>
<tr>
<td>75</td>
<td>0.3</td>
<td>60.5</td>
<td>7,940.6</td>
<td>0.0</td>
<td>6.0</td>
</tr>
<tr>
<td>100</td>
<td>0.4</td>
<td>30.3</td>
<td>9,075.0</td>
<td>0.0</td>
<td>3.0</td>
</tr>
<tr>
<td>125</td>
<td>0.5</td>
<td>0.0</td>
<td>9,453.1</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

* Critical section for shear (see Sect. F.1.11)

Table F.1.4-2. Unfactored Shear Forces and Bending Moments Due to Live Loads for a Typical Interior Beam

<table>
<thead>
<tr>
<th>Distance x, ft</th>
<th>Section x/L</th>
<th>Truck Load with Impact</th>
<th>Lane Load</th>
<th>Fatigue Truck with Impact</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Shear ( V_{LT} ) kips</td>
<td>Moment ( M_{LT} ) ft-kips</td>
<td>Shear ( V_{LL} ) kips</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>87.1</td>
<td>0.0</td>
<td>75.6</td>
</tr>
<tr>
<td>8.53</td>
<td>0.034</td>
<td>84.0</td>
<td>708.6</td>
<td>70.5</td>
</tr>
<tr>
<td>25</td>
<td>0.1</td>
<td>78.0</td>
<td>1,950.5</td>
<td>61.2</td>
</tr>
<tr>
<td>50</td>
<td>0.2</td>
<td>69.0</td>
<td>3,448.8</td>
<td>48.4</td>
</tr>
<tr>
<td>75</td>
<td>0.3</td>
<td>59.9</td>
<td>4,495.0</td>
<td>37.0</td>
</tr>
<tr>
<td>100</td>
<td>0.4</td>
<td>50.9</td>
<td>5,116.8</td>
<td>27.2</td>
</tr>
<tr>
<td>125</td>
<td>0.5</td>
<td>41.8</td>
<td>5,300.6</td>
<td>18.9</td>
</tr>
</tbody>
</table>

F.1.5. ESTIMATE REQUIRED PRESTRESS

The required number of strands is usually governed by concrete tensile stresses at the bottom fiber for load combination Service III at the section of maximum moment or at the harp points. For estimating the number of strands, only the stresses at midspan are considered.

F.1.5.1. Service Load Stresses at Midspan

Bottom tensile stress due to applied dead and live loads using load combination Service III is:

\[
f_b = \frac{M_g + M_s}{S_b} + \frac{M_b + M_{ws} + (0.8)(M_{LT} + M_{LL})}{S_{bc}}
\]

where

- \( f_b \) = concrete tensile stress at bottom fiber of the beam, ksi
- \( M_g \) = unfactored bending moment due to beam self-weight, ft-kips
- \( M_s \) = unfactored bending moment due to slab and haunch weights, ft-kips
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

\[ M_b = \text{unfactored bending moment due to barrier weight, ft-kips} \]
\[ M_{ws} = \text{unfactored bending moment due to future wearing surface, ft-kips} \]
\[ M_{LT} = \text{unfactored bending moment due to truck load, ft-kips} \]
\[ M_{LL} = \text{unfactored bending moment due to lane load, ft-kips} \]

Using values of bending moments from Tables F.1.4-1 and F.1.4-2, bottom tensile stress at midspan is:

\[
\sigma_b = \frac{(9,453.1 + 0) + 937.5 + 1,627.7 + (0.8)(5,300.6 + 4,722.2)}{36,030} \times 12
\]

\[
= 3.148 + 3.525 = 6.67 \text{ ksi}
\]

F.1.5.2. Stress Limits for Concrete

Tensile stress limit at service loads = \( f_{1c} \) (for UHPC) = 1.0 ksi

where \( f_{c'} \) = specified concrete compressive strength of beam for design, ksi

Concrete tensile stress limit = -\( f_{1c} \) (for UHPC) = -1.0 ksi

F.1.5.3. Required Number of Strands

The required precompressive stress at the bottom fiber of the beam is the difference between bottom tensile stress due to applied loads and the concrete tensile stress limit:

\[
\sigma_{pb} = (6.67 - 1) = 5.67 \text{ ksi}
\]

The location of the strand center of gravity at midspan ranges from 5 to 15\% of the beam depth, measured from the bottom of the beam. A value of 5\% is appropriate for newer efficient sections like the bulb-tee beams and 15\% for less efficient AASHTO standard shapes.

Assume the distance between the center of gravity of bottom strands and the bottom fiber of the beam:

\[
y_{bs} = 0.05h = 0.05(108) = 5.4 \text{ in.}, \text{ use } y_{bs} = 4.00 \text{ in.}
\]

Therefore, strand eccentricity at midspan, \( e_c = (y_b - y_{bs}) = (58.34 - 4.00) = 54.34 \text{ in.} \)

If \( P_{pe} \) is the total prestressing force after all losses, the stress at the bottom fiber due to prestress is:

\[
\sigma_{pb} = \frac{P_{pe}}{A_g} \frac{e_c}{S_b} \times 1.127 + \frac{P_{pe}(54.34)}{36,030}
\]

Solving for \( P_{pe} \), the required \( P_{pe} = 2,368.2 \text{ kips} \).

Final prestress force per strand = \( (\text{area of strand})(f_{pi})(1 - \text{losses}) \)

where \( f_{pi} = \text{initial strand stress before transfer, ksi} \) (See Section F.1.2) = 202.5 ksi

Assuming final loss of 25\% of \( f_{pi} \), the prestress force per strand after all losses = (0.294)(202.5)(1 - 0.25) = 44.7 kips

Number of strands required = (2,368.2/44.7) = 52.98 strands

As an initial trial, (54) 0.7-in.-diameter, 270 ksi strands are selected. The center of gravity of the 54 strands at midspan is 4.96 in. from the bottom of the concrete, which is higher than the assumed value, 4.0 in. Thus, a second iteration using a new value of strand eccentricity indicates that 54 strands are required. The strand pattern at midspan for the 54 strands is shown in Figure F.1.5.3-1. Each available position is filled beginning with the bottom row.

Total area of prestressing strands, \( A_{ps} = 54(0.294) = 15.88 \text{ in.}^2 \)

Note: This is a conservative estimate of the number of strands because nontransformed section properties are used in lieu of transformed section properties. The number of strands can be refined later in the design process as more accurate section properties and prestress losses are determined.
F.1.5.4. Strand Pattern

The distance between the center of gravity of bottom strands and the bottom concrete fiber of the beam at midspan is:

\[ y_{bs} = \frac{16(2) + 16(4) + 6(8) + 4(10) + 2(12)}{54} = 4.96 \text{ in.} \]

Strand eccentricity at midspan, \( e_c = y_b - y_{bs} = 58.34 - 4.96 = 53.38 \text{ in.} = e_{pg} \)

F.1.5.5. Steel Transformed Section Properties

For each row of the prestressing strands shown in Figure F.1.5.4-1, the steel area is multiplied by \((n - 1)\) to calculate the transformed section properties, where \(n\) is the modular ratio between prestressing strand and concrete. Since the modulus of elasticity of concrete is different at transfer and final time, the transformed section properties should be calculated separately for the two stages. Using similar procedures as in Section F.1.3.2.3, a sample calculation is shown in Tables F.1.5.5-1, -2, and -3.

At transfer:

\[ n - 1 = \frac{28,500}{5,000} - 1 = 4.70 \]

At final:

\[ n - 1 = \frac{28,500}{6,500} - 1 = 3.38 \]
### Table F.1.5.5-1. Properties of Noncomposite Transformed Section at Transfer

<table>
<thead>
<tr>
<th>Transformed Area, in.²</th>
<th>yₚ in.</th>
<th>Aᵧₚ in.³</th>
<th>A(yₚ₀ₑ₋ yₚ)² in.⁴</th>
<th>I in.⁴</th>
<th>I + A(yₚ₀ₑ₋ yₚ)² in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>1,127</td>
<td>58.34</td>
<td>65,749</td>
<td>12,382</td>
<td>2,102,010</td>
</tr>
<tr>
<td>Row 1</td>
<td>22.109</td>
<td>2</td>
<td>44.22</td>
<td>62,163</td>
<td>62,163.23</td>
</tr>
<tr>
<td>Row 2</td>
<td>22.109</td>
<td>4</td>
<td>88.44</td>
<td>57,562</td>
<td>57,562.35</td>
</tr>
<tr>
<td>Row 3</td>
<td>13.818</td>
<td>6</td>
<td>82.91</td>
<td>33,211</td>
<td>33,211.46</td>
</tr>
<tr>
<td>Row 4</td>
<td>8.291</td>
<td>8</td>
<td>66.33</td>
<td>18,334</td>
<td>18,334.20</td>
</tr>
<tr>
<td>Row 5</td>
<td>5.527</td>
<td>10</td>
<td>55.27</td>
<td>11,205</td>
<td>11,205.23</td>
</tr>
<tr>
<td>Row 6</td>
<td>2.764</td>
<td>12</td>
<td>33.16</td>
<td>5,116</td>
<td>5,115.94</td>
</tr>
<tr>
<td>SUM</td>
<td>1,201.6</td>
<td>66,120</td>
<td></td>
<td></td>
<td>2,301,984</td>
</tr>
</tbody>
</table>

Note: The moment of inertia of strand about its own centroid is neglected.

### Table F.1.5.5-2. Properties of Noncomposite Transformed Section at Final Time

<table>
<thead>
<tr>
<th>Transformed Area, in.²</th>
<th>yₚ in.</th>
<th>Aᵧₚ in.³</th>
<th>A(yₚ₀ₑ₋ yₚ)² in.⁴</th>
<th>I in.⁴</th>
<th>I + A(yₚ₀ₑ₋ yₚ)² in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>1,127</td>
<td>58.34</td>
<td>65,749</td>
<td>12,382</td>
<td>2,102,010</td>
</tr>
<tr>
<td>Row 1</td>
<td>15.921</td>
<td>2</td>
<td>31.84</td>
<td>46,273</td>
<td>46,273.16</td>
</tr>
<tr>
<td>Row 2</td>
<td>15.921</td>
<td>4</td>
<td>63.68</td>
<td>42,904</td>
<td>42,903.53</td>
</tr>
<tr>
<td>Row 3</td>
<td>9.951</td>
<td>6</td>
<td>59.70</td>
<td>24,788</td>
<td>24,788.30</td>
</tr>
<tr>
<td>Row 4</td>
<td>5.970</td>
<td>8</td>
<td>47.76</td>
<td>13,705</td>
<td>13,704.90</td>
</tr>
<tr>
<td>Row 5</td>
<td>3.980</td>
<td>10</td>
<td>39.80</td>
<td>8,390</td>
<td>8,389.72</td>
</tr>
<tr>
<td>Row 6</td>
<td>1.990</td>
<td>12</td>
<td>23.88</td>
<td>3,837</td>
<td>3,837.34</td>
</tr>
<tr>
<td>SUM</td>
<td>1,180.7</td>
<td>66,016</td>
<td></td>
<td></td>
<td>2,248,557</td>
</tr>
</tbody>
</table>

Note: The moment of inertia of strand about its own centroid is neglected.

### Table F.1.5.5-3. Properties of Composite Transformed Section at Final Time

<table>
<thead>
<tr>
<th>Transformed Area, in.²</th>
<th>yₚ in.</th>
<th>Aᵧₚ in.³</th>
<th>A(yₚ₀ₑ₋ yₚ)² in.⁴</th>
<th>I in.⁴</th>
<th>I + A(yₚ₀ₑ₋ yₚ)² in.⁴</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>0</td>
<td>108</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Haunch</td>
<td>0</td>
<td>108</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Beam</td>
<td>1,127</td>
<td>58.34</td>
<td>65,749</td>
<td>12,382</td>
<td>2,102,010</td>
</tr>
<tr>
<td>Row 1</td>
<td>15.921</td>
<td>2</td>
<td>31.84</td>
<td>46,273</td>
<td>46,273.16</td>
</tr>
<tr>
<td>Row 2</td>
<td>15.921</td>
<td>4</td>
<td>63.68</td>
<td>42,904</td>
<td>42,903.53</td>
</tr>
<tr>
<td>Row 3</td>
<td>9.951</td>
<td>6</td>
<td>59.70</td>
<td>24,788</td>
<td>24,788.30</td>
</tr>
<tr>
<td>Row 4</td>
<td>5.970</td>
<td>8</td>
<td>47.76</td>
<td>13,705</td>
<td>13,704.90</td>
</tr>
<tr>
<td>Row 5</td>
<td>3.980</td>
<td>10</td>
<td>39.80</td>
<td>8,390</td>
<td>8,389.72</td>
</tr>
<tr>
<td>Row 6</td>
<td>1.990</td>
<td>12</td>
<td>23.88</td>
<td>3,837</td>
<td>3,837.34</td>
</tr>
<tr>
<td>SUM</td>
<td>1,180.7</td>
<td>66,016</td>
<td></td>
<td></td>
<td>2,248,557</td>
</tr>
</tbody>
</table>

Note: The moment of inertia of strand about its own centroid is neglected.
The transformed section properties are calculated as:

Non composite transformed section at transfer:
\[ A_t = \text{area of transformed section at transfer} = 1,201.6 \text{ in.}^2 \]
\[ I_t = \text{moment of inertia of the transformed section at transfer} = 2,301,984 \text{ in.}^4 \]
\[ e_{ti} = \text{eccentricity of strands with respect to transformed section at transfer} = 50.07 \text{ in.} \]
\[ y_{bti} = \text{distance from the centroid of the transformed section to the extreme bottom fiber of the beam at transfer} = 55.03 \text{ in.} \]
\[ S_{bti} = \text{section modulus for the extreme bottom fiber of the transformed section at transfer} = 41,835 \text{ in.}^3 \]
\[ S_{tti} = \text{section modulus for the extreme top fiber of the transformed section at transfer} = 43,455 \text{ in.}^3 \]

Non composite transformed section at final time:
\[ A_{tf} = \text{area of transformed section at final time} = 1,180.7 \text{ in.}^2 \]
\[ I_{tf} = \text{moment of inertia of the transformed section at final time} = 2,248,557 \text{ in.}^4 \]
\[ e_{tf} = \text{eccentricity of strands with respect to transformed section at final time} = 50.95 \text{ in.} \]
\[ y_{btf} = \text{distance from the centroid of the transformed section to the extreme bottom fiber of the beam at final time} = 55.91 \text{ in.} \]
\[ S_{btf} = \text{section modulus for the extreme bottom fiber of the transformed section at final time} = 40,217 \text{ in.}^3 \]
\[ S_{tf} = \text{section modulus for the extreme top fiber of the transformed section at final time} = 43,167 \text{ in.}^3 \]

Composite transformed section at final time:
\[ A_{tf} = \text{area of transformed composite section at final time} = 1,180.7 \text{ in.}^2 \]
\[ I_{tf} = \text{moment of inertia of the transformed composite section at final time} = 2,248,557 \text{ in.}^4 \]
\[ e_{tf} = \text{eccentricity of strands with respect to transformed composite section at final time} = 50.95 \text{ in.} \]
\[ y_{btf} = \text{distance from the centroid of the composite transformed section to the extreme bottom fiber of the beam at final time} = 55.91 \text{ in.} \]
\[ S_{btc} = \text{section modulus for the extreme bottom fiber of the transformed composite section at final time} = 40,217 \text{ in.}^3 \]
\[ S_{ttc} = \text{composite section modulus for the extreme top fiber of the precast beam for transformed section at final time} = 43,167 \text{ in.}^3 \]
\[ S_{dtc} = \text{composite section modulus for the extreme top fiber of the deck for transformed section at final time} = 73,190 \text{ in.}^3 \]

**F.1.6. PRESTRESS LOSSES**

Total prestress loss:
\[ \Delta f_{PT} = \Delta f_{PSH} + \Delta f_{PSR} + \Delta f_{PCR} + \Delta f_{PR} \]

where

\[ \Delta f_{PSR} = \text{total loss in prestressing steel stress} \]
\[ \Delta f_{PSH} = \text{initial losses due to early-age shrinkage of UHPC} \]
\[ \Delta f_{PCR} = \text{prestress loss due to creep of concrete between time of transfer and final time} \]
\[ \Delta f_{PR} = \text{prestress loss due to relaxation of strands between time of transfer and final time} \]

**F.1.6.1. Early-Age Shrinkage Losses**
\[ \Delta f_{PSH} = \epsilon_{sh} E_p K_d \]
where

\[ \varepsilon_{shi} = \text{concrete autogenous shrinkage strain of girder at early ages} = 600 \text{ microstrain} \]

\[ K_d = \text{transformed section coefficient which may be assumed for elastic (non-creep) effects at initial conditions. Note: this coefficient will be subject to refinement through parametric studies in Phase II.} = 0.830 \]

Therefore, loss due to early-age shrinkage:

\[ \Delta f_{pSHI} = (600 \times 10^{-6})(28,500)(0.83) = 14.19 \text{ ksi} \]

**F.1.6.2. Long-Term Losses**

The total long-term loss between time of transfer and final time is the summation of prestress losses due to shrinkage of concrete, creep of concrete, and relaxation of prestressing strands.

Creep and shrinkage of UHPC are highly related to the method and procedure of curing. For this design example, it is assumed steam curing following the PCI practice.

**Curing Type:** Ambient

Ultimate Shrinkage Strain of Concrete: \( \varepsilon_{SR} = 300 \text{ microstrain} \)

Ultimate Creep Coefficient of Concrete: \( \Psi_{CR} = 1.2 \)

**F.1.6.2.1. Long-Term Losses between Transfer and Final Time**

The following construction schedule is assumed in calculating the long-term losses:

Concrete age at transfer: \( t_i = 1 \text{ day} \)

Concrete age at final stage: \( t_f = 20,000 \text{ days} \)

**F.1.6.2.1.1. Shrinkage of Concrete**

The prestress loss due to shrinkage of concrete between time of transfer and final time is calculated by:

\[ \Delta f_{pSR} = \varepsilon_{bf} E_p K_{if} \]

where

\[ \varepsilon_{bf} = \text{concrete shrinkage strain of girder for time period between transfer and final time} \]

\[ E_p = \text{modulus of elasticity of prestressing strands, ksi} \]

\[ K_{if} = \text{transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between transfer and final time. It is assumed, subject to comprehensive parametric study in Phase II.} = 0.730 \]

The concrete shrinkage strain \( \varepsilon_{bf} \) is taken as:

\[ \varepsilon_{bf} = k_{vs} k_{hs} k_{if} 0.30 \times 10^{-3} \]

(Eq. F.1.6.2.1.1-1)

where

The factor for the effect of the volume-to-surface ratio of the beam:

\[ k_{vs} = 1.000 \text{ for UHPC} \]

The humidity factor for shrinkage:

\[ k_{hs} = 1.000 \text{ for UHPC} \]

The factor for the effect of concrete strength:

\[ k_{if} = 1.000 \text{ for UHPC} \]

The time development factor at final time: 
\[ k_{tf} = 1.000 \text{ for UHPC} \]

\[ \varepsilon_{bf} = (1.000)(1.000)(1.000)(0.30 \times 10^{-6}) = 0.0003 \]

The prestress loss due to shrinkage of concrete between transfer and final time is:

\[ \Delta f_{psR} = (0.0003)(28,500)(0.73) = 6.24 \text{ ksi} \]

**F.1.6.2.1.2. Creep of Concrete**

The prestress loss due to creep of girder concrete between time of transfer and final time is:

\[ \Delta f_{pCR} = \frac{E_p}{E_{ct}} f_{spp} \Psi_b(t_f, t_i) K_{tf} \]

where

- \( E_{ct} \) = modulus of elasticity of beam concrete at transfer = 5,000 ksi
- \( f_{spp} \) = sum of concrete stresses at the center of gravity of prestressing strands due to prestressing force at transfer and the self-weight of the member at sections of maximum moment
- \( \Psi_b(t_f, t_i) \) = girder creep coefficient at final time due to loading introduced at transfer

The prestress force per strand before transfer = (area of strand)(prestress stress before transfer) = (0.294)(202.5) = 59.50 kips

\[ f_{spp} = \frac{P_{pi}}{A_{ti}} + \frac{P_{pi} e_{ti}^2}{I_{ti}} - \frac{M_g e_{ti}}{I_{ti}} \]

where

- \( P_{pi} \) = total prestressing force before transfer = (54)(59.5) = 3,213 kips
- \( M_g \) = bending moment due to self-weight of the beam. It should be calculated based on the overall beam length.

Since the elastic shortening loss is a part of the total loss, \( f_{spp} \) will be conservatively computed based on \( M_g \) using the design span.

\[ f_{spp} = \frac{3,213}{1,201.6} + \frac{(3,213)(50.07)^2}{2,301,984} - \frac{(9,453.1)(12)(50.07)}{2,301,984} = 3.71 \text{ kips} \]

\[ \Psi_b(t_f, t_i) = 1.2 k_{sc} k_{bc} k_{hf}^t (1.000) \]  

where \( k_{sc} \) = the humidity factor for creep = 1.000 for UHPC

Therefore

\[ \Psi_b(t_f, t_i) = 1.2(1.000)(1.000)(1.000)(1.000)(1.000) = 1.20 \]

\[ \Delta f_{pCR} = \frac{28,500}{5,000} (3.71)(1.2)(0.73) = 18.52 \text{ ksi} \]

**F.1.6.2.1.3. Relaxation of Prestressing Strands**

According to LRFD Art. 5.9.5.4.2c, the relaxation loss between transfer and deck placement may also be assumed equal to 1.20 ksi for low-relaxation strands, and between deck placement and final time = 1.20 ksi. Total relaxation loss for this example may be assumed = 2.40 ksi.

\[ \Delta f_{pR} = 2.40 \text{ ksi} \]

**F.1.6.2.2. Total Time-Dependent Loss**

The total time-dependent loss, \( \Delta f_{ptT} \), is determined as:

\[ \Delta f_{ptT} = \Delta f_{pSHI} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR} \]  

[LRFD Eq. 5.9.3.4.1-1]
When determining the concrete stress using transformed section properties, all the elastic losses and gains are implicitly accounted for.

\[ f_{pe} = f_{pi} - \Delta f_{plT} = 202.5 - 41.35 = 161.15 \text{ ksi} \]

Force per strand without live load gains = \( f_{pe} \cdot \text{(area of strand)} = 161.15(0.294) = 47.38 \text{ kips} \)

Therefore, the total prestressing force after losses, \( P_{pe} = 47.38 \cdot 54 = 2,559 \text{ kips} \)

If one assumes that the beam was thermally cured per UHPC curing specifications, the ultimate shrinkage strain (excluding initial autogenous shrinkage) would be zero and the creep coefficient would be 0.3. The resulting total long-term prestress loss would be:

\[
\Delta f_{plT} = \Delta f_{pSHI} + \Delta f_{pSR} + \Delta f_{pcR} + \Delta f_{pR} \quad \text{[LRFD Eq. 5.9.3.4.1-1]}
\]

\[ = 14.19 + 0 + 4.63 + 2.4 \]

\[ = 14.19 + 7.03 = 21.22 \text{ ksi} \]

Which is about 50% reduction in prestress loss compared to conventional MNL 116 curing specifications.

The analysis in this example will continue with the values corresponding to conventional curing.

**F.1.7. CONCRETE STRESSES AT TRANSFER**

**F.1.7.1. Stress Limits for Concrete**

Compression:
- \( 0.70 f_{ci}' = 0.7(10) = 7.0 \text{ ksi} \)

Tension:
- without bonded auxiliary reinforcement
  - For UHPC, \( f_{ti} = -0.75 \text{ ksi} \)
  - with bonded auxiliary reinforcement that is sufficient to resist 120% of the tension force in the cracked concrete
    - For UHPC, \( f_{ti} = -0.75 \text{ ksi} \)

**F.1.7.2. Stresses at Transfer Length Section**

Stresses at this location need only be checked at transfer because this stage almost always governs. Also, losses with time will reduce the concrete stresses making them less critical.

Transfer length = 20(strand diameter) = 20(0.7) = 14 in. = 1.2 ft \[ \text{[LRFD Art. 5.9.4.3]} \]

Due to camber of the beam at transfer, the beam self-weight acts on the overall beam length, 252-ft. Therefore, values for bending moment given in Table F.1.4-1 cannot be used because they are based on the design span 250-ft. Using statics, bending moment at transfer length due to beam self-weight is:

\[ M_g = 0.5 w_g x (L - x) = (0.5)(121)(14/12)(252 - (14/12)) = 177.05 \text{ ft-kips} \]

Compute stress in the top of beam:

\[ f_t = \frac{P_{pl} - P_{pl} e_t}{s_{tti}} + \frac{M_g}{s_{tti}} = \frac{3,213}{1,201.6} - \frac{3,213(50.07)}{43,455} + \frac{177.05(12)}{43,455} \]

\[ = 2.67 - 3.70 + 0.05 = -0.98 \text{ksi} \]

Tensile stress limit for concrete with bonded reinforcement: -0.75 ksi \( \text{NG} \)

Compute stress in the bottom of beam:
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

\[ f_b = \frac{P_{pl}}{A_{tt}} + \frac{P_{pl}e_{ti}}{S_{btt}} - \frac{M_g}{S_{btt}} = \frac{3,213}{1,201.6} + \frac{3,213(50.07)}{41,835} - \frac{177.05(12)}{41,835} \]
\[ = 2.67 + 3.85 - 0.05 = 6.47 \text{ ksi} \]

Compressive stress limit: +7.00 ksi OK

Since the stresses at the top exceed the stress limits, debond strands to satisfy the specified limits. Try debonding 14 strands from each end. The number of debonded strands in each row can be seen in Table F.1.7.2-1.

**Table F.1.7.2-1. Strand Pattern**

<table>
<thead>
<tr>
<th>Row</th>
<th>No. Strands</th>
<th>Distance from bottom (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>5</td>
<td>16</td>
<td>4</td>
</tr>
<tr>
<td>6</td>
<td>16</td>
<td>2</td>
</tr>
</tbody>
</table>

At midspan

The distance between the center of gravity of bottom strands and the bottom concrete fiber of the beam at beam end is:

\[ y_{bs} = \frac{2(12) + 4(10) + 6(8) + 8(6) + 10(4) + 10(2)}{40} = 5.50 \text{ in.} \]

The distance between the center of gravity of all strands and the bottom concrete fiber of the beam at transfer length is:

\[ y_{bs} = \frac{10(2) + 10(4) + 8(6) + 6(8) + 4(10) + 2(12)}{40} = 5.50 \text{ in.} \]

Eccentricity of the 54-strand group at transfer length, \( e_{tb} \), is: \( y_{bt} - y_{bs} = 55.03 - 5.50 = 49.53 \text{ in.} \)

Recompute stress in the top of the beam at the transfer length section with debonded strands.

\[ f_t = \frac{P_{pl}}{A_{tt}} - \frac{P_{pl}e_{ti}}{S_{ttt}} + \frac{M_g}{S_{ttt}} = \frac{2,380}{1,201.6} + \frac{2,380(49.53)}{43,455} - \frac{177.05(12)}{43,455} \]
\[ = 1.98 - 2.71 + 0.05 = -0.68 \text{ ksi} \]

Tensile stress limit for concrete with bonded reinforcement: -0.75 ksi OK

Compute stress in the bottom of beam:

\[ f_b = \frac{P_{pl}}{A_{tt}} + \frac{P_{pl}e_{ti}}{S_{btt}} - \frac{M_g}{S_{btt}} = \frac{2,380}{1,201.6} + \frac{2,380(49.53)}{41,835} - \frac{177.05(12)}{41,835} \]
\[ = 1.98 + 2.82 - 0.05 = 4.75 \text{ ksi} \]

Compressive stress limit: +7.00 ksi OK

**F.1.7.3. Stresses at Midspan**

Bending moment at midspan due to the beam self-weight is:

\[ M_g = 0.5w_gx(L - x) = (0.5)(1.21)(126)[252 - 126] = 9,604.98 \text{ ft-kips} \]
Concrete stress in the top of beam:

\[ f_t = \frac{P_{pe}}{A_{tt}} - \frac{P_{pe}e_{ti}}{S_{tti}} + \frac{M_g}{S_{tti}} = \frac{3.213}{1,201.6} - \frac{3.213(50.07)}{43,455} + \frac{9,604.98(12)}{43,455} \]
\[ = 2.67 - 3.70 + 2.65 = 1.62 \text{ ksi} \]

Tensile stress limit: -0.75 ksi  \hspace{1cm} \text{OK}

Compute stress in the bottom of beam:

\[ f_b = \frac{P_{pe}}{A_{tt}} + \frac{P_{pe}e_{ti}}{S_{btf}} - \frac{M_g}{S_{bti}} = \frac{3.213}{1,201.6} + \frac{3.213(50.07)}{41,835} - \frac{9,604.98(12)}{41,835} \]
\[ = 2.67 + 3.85 - 0.05 = 3.76 \text{ ksi} \]

Compressive stress limit: +7.00 ksi  \hspace{1cm} \text{OK}

### F.1.8. CONCRETE STRESSES AT SERVICE LOADS

Using transformed section properties and refined losses, \( P_{pe} = 2,620.0 \text{ kips} \)

#### F.1.8.1. Stress Limits for Concrete

**Compression:**

Due to permanent loads, (i.e. beam self-weight, weight of slab and haunch, weight of future wearing surface, and weight of barriers), for load combination Service I:

- for precast beams: \( 0.6 f'_c = 0.6(18.0) = 10.8 \text{ ksi} \)

Due to permanent and transient loads (i.e. all dead loads and live loads), for load combination Service I:

- for precast beams: Not required for UHPC

**Tension:**

For components with bonded prestressing tendons for Service III:

- for precast beams: \(-1.00 \text{ ksi} = -f_i\)

#### F.1.8.2. Stresses at Midspan

**F.1.8.2.1. Concrete Stress at Top Fiber of the Beam**

To check top compressive stresses, two cases are considered:

1. Under permanent loads, load combination Service I:

   Using bending moment values given in Table F.1.4-1, compute the top fiber stresses:

   \[ f_{tg} = \frac{P_{pe}}{A_{tf}} - \frac{P_{pe}e_{tf}}{S_{ttf}} + \frac{M_g}{S_{tti}} + \frac{M_s + M_{ws} + M_b}{S_{ttf}} \]
   \[ = \frac{2,559}{1,180.7} - \frac{2,559(50.95)}{43,167} + \frac{9,453.1(12)}{43,455} + \frac{(0 + 1,627.7 + 937.5)(12)}{43,167} \]
   \[ = 2.17 - 3.02 + 2.65 + 0.71 = 2.51 \text{ ksi} \hspace{1cm} \text{OK} \]

2. Under permanent and transient loads, load combination Service I:

   Check is not required for UHPC

**F.1.8.2.2. Concrete Stress in Bottom of Beam, Load Combination Service III**

Using bending moment values given in Table F.1.4-1, compute the bottom fiber stresses:

\[ f_{b} = \frac{P_{pe}}{A_{bf}} + \frac{P_{pe}e_{bf}}{S_{btf}} - \frac{M_g}{S_{bti}} - \frac{M_s - M_{ws} + M_b - 0.8(M_{LT} + M_{LL})}{S_{btf}} \]
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Appendix F - Design Examples of Bridge Beams

\[
\frac{2,559}{1,180.7} + \frac{(2,559)(50.95)}{40,217} - \frac{(9,453.1)(12)}{41,835} - \frac{[0 - (1,627.7 + 937.5) - 0.8(5,300.6 + 4,722.2)](12)}{40,217}
\]

\[
= 2.17 + 3.24 - 2.76 - 0 - 0.77 - 2.39 = -0.51 \text{ ksi}
\]

Tensile stress limit: -1.00 ksi  OK

F.1.8.3. Fatigue Stress Limit
Not required for UHPC

F.1.8.4. Summary of Stresses at Midspan at Service Loads

<table>
<thead>
<tr>
<th>Top of Beam, ksi</th>
<th>Bottom of Beam, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service I</td>
<td>Service III</td>
</tr>
<tr>
<td>Permanent Loads</td>
<td>Total Loads</td>
</tr>
<tr>
<td>2.51</td>
<td>5.30</td>
</tr>
</tbody>
</table>

F.1.9. FLEXURAL STRENGTH

Total ultimate bending moment for Strength I is:

\[
M_u = 1.25(\Delta C) + 1.5(\Delta W) + 1.75(\Delta L + \Delta M)
\]

Using the values of unfactored bending moment given in Tables F.1.4-1 and F.1.4-2, the ultimate bending moment at midspan is:

\[
M_u = 1.25(9,453.1 + 0 + 937.5) + 1.5(1,627.7) + 1.75(5,300.6 + 4,722.2) = 32,969.70 \text{ ft-kips}
\]

In order to maintain equilibrium, the sum of the tension and compression forces must equal zero. The process is iterative by assuming the distance from the extreme compression fiber to the neutral axis \(c\). Assume rectangular section behavior and check if the depth of the equivalent compression stress block, \(a\), is less than or equal to \(t_s\):

\[
\beta_1 = \text{stress factor of compression block} \quad \text{[LRFD Art. 5.6.2.2]}
\]

\[
\beta_1 = 0.85 \text{ for } f_c' \leq 4.0 \text{ ksi}
\]

\[
\beta_1 = 0.85 - 0.05(f_c' - 4.0) \geq 0.65 \text{ for } f_c' > 4.0 \text{ ksi}
\]

Therefore,

\[
\beta_1 = 0.85 - 0.05(18.0 - 4.0) = 0.15 < 0.65, \text{ use 0.65}
\]

\[
a = (0.65)(3.99) = 2.59 \text{ in.}
\]

\[
a > h_f = 2 \text{ in.}
\]

Therefore, the rectangular section behavior assumption is NOT valid

The section should be modeled as a T-section introducing the effect of changing the width of the compression flange with depth as shown in Figure F.1.9-1.
Use:

\[ c = 6.07 \text{ in.} \]
\[ a = 3.95 \text{ in.} < t_1 + t_2 = 5 \text{ in.} \]

Therefore, the Tee section behavior assumption is valid.

The strand layers may be represented by a group with the same stress-strain properties and effective prestress. This group has a combined area with a single centroid \(d_p\). The strain in the group is:

\[ \varepsilon_s = \varepsilon_c \left( \frac{d_p}{c} - 1 \right) + \left( \frac{f_{ps}}{E_p} \right) \]

where

\[ \varepsilon_c = \text{ultimate compressive strain} = 0.003 \]
\[ d_p = \text{distance from extreme compression fiber to the centroid of the prestressing strands} \]
\[ = h + \text{Haunch Thickness} + ts - y_{bs} \]
\[ = 108 + 0 + 4.96 = 103.04 \text{ in.} \]
\[ f_{ps} = \text{effective prestress, ksi} = 161.15 \text{ ksi} \]

Therefore,

\[ \varepsilon_s = 0.003 \left( \frac{103.04}{3.99} - 1 \right) + \left( \frac{161.15}{28,500} \right) = 0.08 \]

Check the equilibrium of forces:

\[ \Sigma F = 0.85f_c' \beta_1 c b_w + 0.85f_c'(b - b_w)h_f + A_{ps}f_{ps} \]

where

\[ f_c' = \text{specified compressive strength of compression block} = 18.0 \text{ ksi} \]
\[ b = \text{effective width of compression flange} = 108.00 \text{ in.} \]
\[ b_w = \text{width of stem in compression zone} = 33 \text{ in.} \]
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

\[ h_f = \text{depth of compression flange} = 2 \text{ in.} \]
\[ A_{ps} = \text{area of prestressing strand} \]
\[ = (54)(0.294) = 15.88 \text{ in}^2 \]
\[ f_{ps} = \text{average stress in prestressing strand, ksi} \]
\[ = \epsilon_s (887 + \frac{27613}{1 + (112.4 \epsilon_s)^{7.36}}) \leq 270.0 \text{ ksi} \quad \text{[BDM Eq. 8.2.5-1]} \]
\[ = 0.079 (887 + \frac{27613}{1 + (112.4 (0.079))^{7.36}}) \leq 270.0 \text{ ksi} \]
\[ = 315.74 \text{ ksi} > 270.0 \text{ ksi}, \text{ use } f_{ps} = 270.0 \text{ ksi} \]

Sum of compression and tension forces:
\[ \sum F = -0.85 (18)(0.65)(6.07)(33) - (0.85)(18)(108 - 33)(2) + (15.88)(270) = 0 \]

Nominal flexural resistance:
\[ M_n = A_{ps} f_{ps} \left( d_p - \frac{a}{2} \right) + 0.85 f'_c (b - b_w) h_f \left( \frac{a}{2} - \frac{h_f}{2} \right) \quad \text{[LRFD Eq. 5.6.3.2.1-1]} \]

The above equation is a simplified form of LRFD Equation 5.6.3.2.1-1 because no compression reinforcement or nonprestressed reinforcement is considered.
\[ M_n = \left[ (15.88)(270) \left( 103.04 - \frac{3.95}{2} \right) + (0.85)(18)(108 - 33)(2) \left( \frac{3.95}{2} - \frac{2}{2} \right) \right] / 12 = 36,296.99 \text{ ft-kips} \]

Factored flexural resistance:
\[ M_r = \phi M_n \quad \text{[LRFD Eq. 5.6.3.2.1-1]} \]

where
\[ \phi = \text{resistance factor} \quad \text{[LRFD Art. 5.5.4.2]} \]
\[ = 1.00, \text{ for tension controlled prestressed concrete sections} \]

Adequate ductility of the beam is ensured by evaluating whether the member can be classified as tension-controlled. If the member does not satisfy the requirements to be tension-controlled, the resistance factor for the strength limit state 1 check will be reduced in accordance with LRFD Article 5.5.4.2.
\[ 0.75 \leq \phi = 0.75 + \frac{0.25(\epsilon_t - \epsilon_{cl})}{(\epsilon_{tl} - \epsilon_{cl})} \leq 1.0 \quad \text{[LRFD Eq. 5.5.4.2-1]} \]

where
\[ \epsilon_t \quad \text{net tensile strain in the extreme tension steel at nominal resistance} \]
\[ = \epsilon_c \left( \frac{d_p}{c} - 1 \right) \]
\[ = 0.003 \left( \frac{103.04}{3.99} - 1 \right) = 0.074 \]
\[ \epsilon_{cl} \quad \text{compression-controlled strain limit in the extreme tension steel} = 0.002 \quad \text{[LRFD Art. 5.6.2.1]} \]
\[ \epsilon_{tl} \quad \text{tension-controlled strain limit in the extreme tension steel} = 0.005 \quad \text{[LRFD Art. 5.6.2.1]} \]

Therefore,
\[ \phi = 0.75 + \frac{0.25(0.074 - 0.002)}{0.005 - 0.002} \]
\[ = 6.75 > 1.0, \text{ use } \phi = 1.00 \]
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

\[ M_r = (1.00)(36,296.99) = 36,296.99 \text{ ft-kips} > M_u = 32,969.70 \text{ ft-kips} \] OK

**F.1.10. MINIMUM FLEXURE REINFORCEMENT**

At any section, the amount of prestressed and nonprestressed tensile reinforcement must be adequate to develop a factored flexural resistance, \( M_r \), equal to the lesser of:

- 1.33 times the factored moment required by the applicable strength load combination
- The cracking moment determined as follows:

Check at midspan:

\[ M_{cr} = \gamma_3 \left[ (\gamma_1 f_r + \gamma_2 f_{cpe}) S_c - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \right] \]  

[LRFD Eq. 5.6.3.3-1]

where

- \( f_r \) = specified peak flexural strength, \( f_{it} = 1.5 \text{ ksi} \)
- \( f_{cpe} \) = compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads
- \( S_{bcf} = 40,217 \text{ in.}^3 \)

\[ M_{dnc} = \text{noncomposite dead load moment at the section} \]
\[ M_{u} = M_g + M_s = 9,453.10 + 0 = 9,453.10 \text{ ft-kips} \]
\[ S_c = \text{section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads} \]
\[ S_{nc} = \text{section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads} \]
\[ \gamma_1 = \text{flexural cracking variability factor} = 1.6 \]
\[ \gamma_2 = \text{prestress variability factor} = 1.1 \]
\[ \gamma_3 = \text{ratio of specified minimum yield strength to ultimate tensile strength of the nonprestressed reinforcement} = 1 \]

Therefore,

\[ M_{cr} = (1) \left[ \left( \frac{(1.6)(1.5) + (1.1)(5.52)}{12} \right) \frac{40,217}{12} - 9,453.10 \left( \frac{40,217}{40,217} - 1 \right) \right] = 28,393.20 \text{ ft-kips} \]

At midspan, the factored moment required by the Strength I load combination is:

\[ M_u = 32,969.70 \text{ ft-kips} \] (from Section F.1.9)

Thus, \( 1.33 M_u = 1.33(32,969.70) = 43,849.70 \text{ ft-kips} \)

Since \( M_{cr} = 28,393.20 \text{ ft-kips} < 1.33 M_u = 43,849.70 \text{ ft-kips} \), the \( M_{cr} \) requirement controls

\[ M_r = 36,296.99 \text{ ft-kips} > M_{cr} = 28,393.20 \text{ ft-kips} \] OK

Note: The LRFD Specifications requires that this criterion be met at every section.

**F.1.11. SHEAR DESIGN**

The area and spacing of shear reinforcement, if any, must be determined at regular intervals along the entire length of the beam. In this design example, transverse shear design procedures are demonstrated below by determining these values at the critical section near the supports.

Transverse shear reinforcement is required when:

\[ eConstruc\text{-}WJE\text{-}UNL\text{-}NCSU \]
\( V_u > \phi (V_c + V_f + V_p) \)  

where  

\( V_u \) = total factored shear force, kips  
\( V_c \) = nominal shear resistance provided by tensile stresses in the concrete, kips  
\( V_f \) = nominal shear resistance provided by tensile stresses in the fibers, kips  
\( V_p \) = component in the direction of the applied shear of the effective prestressing force, kips  
\( \phi \) = resistance factor = 0.9 

[Modified LRFD Eq. 5.7.2.3-1]  

**F.1.11.1. Critical Section**  
The critical section near the supports is taken as the effective shear depth, \( d_e \), from the internal face of the support.  

\[ d_e = \text{distance between resultants of tensile and compressive forces, } (d_e - \frac{a}{2}) \text{, but not less than } (0.9 d_e) \text{ or } (0.72 h_c) \]  

[LRFD Art. 5.7.2.8]  

where  

\( d_e \) = the corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement  
\( a \) = depth of compression block = 2.59 in. at midspan (assumed adequate)  
\( h_c \) = overall depth of the composite section = 108.00 in.  

Since some of the strands are debonded, the effective depth, \( d_e \), varies from point-to-point. However, \( d_e \) must be calculated at the critical section in shear which is not yet determined; therefore, for the first iteration, \( d_e \) is calculated based on the center of gravity of the straight strand group at the end of the beam, \( y_{bs} \).  

\[ d_e = h_c - y_{bs} = 108.00 - 5.50 = 102.50 \text{ in.} \]  
\[ d_f = 102.5 - (2.59/2) = 101.21 \text{ in.} \]  
\[ > 0.9d_e = 0.9(102.5) = 92.25 \text{ in.} \]  
\[ > 0.72h_c = 0.72(108) = 77.76 \text{ in.} \]  

Therefore, \( d_e = 101.21 \text{ in.} \).  

Because the width of the bearing is not yet determined, it is conservatively assumed to be zero. Therefore, the critical section in shear is located at a distance of:  

101.21 in. = 8.43 ft from centerline of support  
\( (x/L) = 8.43/252 = 0.034L \)  

The effective depth, \( d_e \), and the position of the critical section in shear may be refined based on the position of the critical section calculated above. However, the difference is small. Therefore, no more refinement is performed.  

**F.1.11.2. Contribution of Concrete to Nominal Shear Resistance**  
The contribution of the concrete to the nominal shear resistance is:  

\[ V_c = 0.0316\beta \sqrt{f_c' b_v d_f} \]  

[LRFD Eq. 5.7.3.3-3]  

where  

\( \beta \) = a factor indicating the ability of diagonally cracked concrete to transmit tension (a value indicating concrete contribution).  

Several quantities must be determined before this expression can be evaluated.  

**F.1.11.2.1. Strain in Flexural Tension Reinforcement**  
Calculate the strain at the centroid of the tension reinforcement, \( \varepsilon_s \)
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

\[ \varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0.5N_u + |(V_u - V_p)| - A_psf_{po}}{(E_sA_s + E_pA_{ps})} \quad \text{[LRFD Eq. 5.7.3.4.2-4]} \]

where

- \( M_u \) = applied factored bending moment at the critical section
- \( N_u \) = applied factored axial force at the critical section = 0 kips
- \( V_u \) = applied factored shear force at the critical section
- \( V_p \) = (Force per strand without live load gains)(Number of harped strands)(sin \( \psi \)) = 0
- \( A_{ps} \) = area of prestressing strands on the flexural tension side of the member = 54(0.294) = 15.88 in.\(^2\)
- \( f_{po} \) = a parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete (ksi).
  - For pretensioned members, LRFD Article 5.7.3.4.2 indicates that \( f_{po} \) can be taken as 0.7\( f_{pu} \).
  - (Note: use this for both pretensioned and post-tensioned systems made with stress relieved and low relaxation strands).
    \[ = 0.7(270) = 189.0 \text{ ksi} \]

Therefore,

\[ \varepsilon_s = \frac{\frac{|4,329.33|}{102.21} + 0.05(0) + |(500.6 - 0)| - (15.88)(189)}{(0 + 28,500(15.88))} = -4.391 \times 10^{-3} \]

\( \varepsilon_s \) is less than zero. Use \( \varepsilon_s = 0 \)

F.1.11.2.2. Values of \( \beta \) and \( \theta \)

Assume the section contains at least the minimum amount of transverse reinforcement:

\[ \beta = \frac{4.8}{(1 + 750\varepsilon_s)} = \frac{4.8}{(1 + 750(0))} = 4.8 \quad \text{[LRFD Eq. 5.7.3.4.2-1]} \]

Angle of diagonal compressive stresses is:

\[ \theta = 29 + 3,500\varepsilon_s = 29 + 3,500(0) = 29.0^\circ \quad \text{[LRFD Eq. 5.7.3.4.2-3]} \]

F.1.11.2.3. Compute Concrete Contribution

The nominal shear resisted by the concrete is:

\[ V'_c = 0.0316\beta\sqrt{f'c}b vd_v \quad \text{[LRFD Eq. 5.7.3.3-3]} \]

where \( b_v \) = effective web width = 4.00 in.

\[ V'_c = 0.0316(4.8)\sqrt{18.0}(4)(101.21) = 260.5 \text{ kips} \]

F.1.11.3. Contribution of Fiber to Nominal Shear Resistance

The nominal shear resisted by the fiber is:

\[ V'_f = \psi f_{fu} \cot \theta b vd_v \]

where

\[ \psi f_{fu} = \text{residual rupture stress} = 0.75 \text{ ksi} \]

Therefore,
\[ V_f = (0.75) \cot(29.0^\circ) (4)(101.21) = 547.8 \text{ kips} \]

**F.1.11.4. Contribution to Transverse Reinforcement to Nominal Shear Resistance**

Check if \( V_u > \Phi (V_c + V_f + V_p) \)

\[ \Phi (V_c + V_f + V_p) = (0.9)(260.5 + 547.8 + 0) = 727.5 \text{ kips} > 500.6 \text{ kips} \quad \text{OK} \]

Therefore, transverse shear reinforcement is not needed.

**F.1.11.5. Maximum Nominal Shear Resistance**

In order to ensure that the concrete in the web of the beam will not crush prior to pull out of the fibers, AFGC (2013) gives an upper limit of \( V_n \) as the ultimate strength of the compressive struts as follows:

\[ V_n = 0.18 f'_c b_v d_v + V_p \]

Comparing this equation with LRFD Eq. 5.7.3.3-2, it can be concluded that:

\[
\begin{align*}
V_c + V_s + V_f & \text{ must not be greater than } V_n = 0.18 f'_c b_v d_v \\
V_c + V_s + V_f & = 260.5 + 0 + 547.8 = 808.3 \text{ kips} \\
V_n & = 0.18 f'_c b_v d_v = 0.18 (18.0)(4)(101.21) = 1,311.7 \text{ kips} \\
V_n & = 1,311.7 \text{ kips} > V_c + V_f + V_p = 808.3 \text{ kips} \quad \text{OK}
\end{align*}
\]

Using the above procedures, the transverse reinforcement can be determined at increments along the entire length of the beam.

**F.1.12. Minimum Longitudinal Reinforcement Requirement**

Longitudinal reinforcement should be proportioned so that at each section the following equation is satisfied:

\[
A_{ps} f_{ps} + A_s f_y \geq \frac{M_u}{d_v \Phi_f} + 0.5 \frac{N_u}{\Phi_c} + \left( \frac{|V_u - V_p|}{\Phi_v} - 0.5 V_p \right) \cot \theta
\]

where

- \( A_s \) = area of nonprestressed tension reinforcement = 0.00 in.\(^2\)
- \( f_y \) = specified yield strength of reinforcing bars = 60.0 ksi
- \( A_{ps} \) = area of prestressing strand at the tension side of the section = (40)(0.294) = 11.76 in.\(^2\)
- \( f_{ps} \) = average stress in prestressing strand at the time for which the nominal resistance is required
- \( M_u \) = factored moment at the section corresponding to the factored shear force = 0.00 ft-kips
- \( N_u \) = applied factored axial force = 0.0 kips
- \( V_u \) = factored shear force at face of support = 531.0 kips
- \( V_s \) = shear resistance provided by shear reinforcement = 0.0 kips
- \( V_p \) = component in the direction of the applied shear of the effective prestressing force = 0.0 kips
- \( d_v \) = effective shear depth = 101.21 in
- \( \Phi \) = resistance factor as appropriate for moment, shear, and axial resistance. Therefore, different \( \Phi \) factors will be used for the terms in LRFD Eq. 5.7.3.5-1, depending on the type of action being considered.

Using the above equation, the minimum required reinforcement can be determined at increments along the entire length of the beam. The following calculations present the required reinforcement at face of bearing.
Figure F.1.12-1 Location of shear points and $T_{critical}$

Assume bearing length = 6.00 in.
Embedment of bonded bottom strands = $12 + 0.50(6) + 5.5\cot(29°) = 24.9$ in.
Since the transfer length is 14-in. from the end of the beam, the available prestress from the 40 bonded strands is the same as the effective prestress, $f_{ps}$ in these strands.
Therefore, the available tension tie force is:
$$A_{ps}f_{ps} + A_{s}f_{y} = (40)(0.294) \left(161.15 \left(\frac{24.9}{14}\right) + (0)(60)\right) = 3,371 \text{ kips}$$
And the actual force is:
$$\frac{M_{u}}{d_{v}\phi_{f}} + 0.5\frac{N_{u}}{\phi_{c}} + \left(\frac{V_{u} - V_{p}}{\phi_{v}} - 0.5V_{p}\right)\cot \theta$$
$$= \frac{0(12)}{(101.21)(0.9)} + 0.5\frac{0}{0.65} + \left(\frac{500.6}{0.9} - 0 - 0.5(0)\right)\cot (29°) = 1,003.5 \text{ kips}$$
The provided longitudinal reinforcement is satisfactory.

**F.1.13. PRINCIPAL STRESSES**
The principal tensile stresses shall be checked to not exceed the allowable stresses at the Service III limit state at any section along the girder length. The principal tensile stresses shall be determined using the combination of axial and shear stress. The critical shear design section may be considered as the most critical section for tensile principal stresses. The following calculations show the principal stresses in the web just below the top flange fillet. However, other locations along the web height should be checked too.

The distance of this location from the bottom fibers is 92 in.
Principal tensile stresses may be determined based on the analysis using Mohr’s Circle, where
$$f_{min} = \frac{1}{2} \left( f_{pcx} + f_{pcy} \right) - \sqrt{\left( f_{pcx} + f_{pcy} \right)^2 + \left( 2\tau \right)^2}$$
[LRFD Eq. 5.9.2.3.3-4]
where
$$f_{min} = \text{minimum principal stress in the web, tension negative, ksi}$$
$$f_{pcx} = \text{horizontal stress in the web, ksi}$$
$$f_{pcy} = \text{vertical stress in the web = 0.0 ksi}$$
$$\tau = \text{shear stress, ksi}$$
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

Using bending moment values given in Tables F.1.4-1 and F.1.4-2, compute the normal stresses in the web just below the top flange at section $d_f$ from CL of support. The horizontal stress in the web $f_{pcx}$ is taken as:

$$f_{pcx} = \frac{f_{pe} A_{ps}}{A_{tf}} - \frac{f_{pe} A_{ps} e_{tf}}{l_{tf}} y_{tf} + \frac{M_g}{l_{ti}} y_{ti} + \frac{(M_{ws} + M_p)}{l_{tf}} y_{tf} + \frac{0.8(M_{LT} + M_{LL})}{l_{tf}} y_{tf}$$

where

- $f_{pi}$ = initial strand stress before transfer = 202.5 ksi
- $A_{ps}$ = area of bonded strands crossing the section under investigation, in.$^2$
- $l_{ti}$ = moment of inertia of the transformed section at transfer = 2,301,984 in.$^4$
- $e_{tf}$ = eccentricity of strands with respect to transformed section at final time = 50.95 in.
- $y_{tf}$ = distance from the centroid of the transformed section to the extreme top fiber of the web just below the top flange at transfer

$$y_{tf} = 92 - 55.03 = 36.97 \text{ in.}$$

- $A_{tf}$ = area of transformed section at transfer = 1,180.7 in.$^2$
- $l_{tf}$ = moment of inertia of the transformed section at final time = 2,448,868 in.$^4$
- $y_{tf}$ = distance from the centroid of the transformed section to the extreme top fiber of the web just below the top flange at final time

$$y_{tf} = 92 - 58.34 = 33.66 \text{ in.}$$

Therefore,

$$f_{pcx} = \frac{(161.15)(14.11)}{0.08(708.6 + 615.7)(12)(33.66)} - \frac{(161.15)(14.11)(50.95)(33.66)}{2,448,868} + \frac{1,232.6(12)(36.97)}{2,301,984} + \frac{212.2 + 122.2}{2,448,868} + \frac{122.2}{2,448,868}$$

$$= 1.93 - 1.75 + 0.24 + 0.06 + 0.17 = 0.65 \text{ ksi}$$

Using shear values given in Tables F.1.4-1 and F.1.4-2, compute the shear stress in the web just below the top flange at section $d_f$ from CL of the support. The shear stress, $\tau$, for an open section, is considered as a thin walled section and is taken as:

$$\tau = \frac{V Q_s}{I_p b_w}$$

where

- $V$ = shear force for Service III load combination

$$V = 141 + 0 + 24.3 + 14 + 0.8(70.5 + 84) = 302.9 \text{ kips}$$

- $Q_s$ = the first moment about the neutral axis of gross concrete area above or below the height of the web where the principal tension is being checked = 17,989 in.$^3$

- $I_p$ = moment of inertia of the gross concrete section about the centroidal axis, neglecting the reinforcement = 2,102,010 in.$^4$

- $b_w$ = web width at height of the web where principal tension is being checked = 4 in.

Therefore,

$$\tau = \frac{(302.9)(17,989)}{(2,102,010)(4)} = 0.65 \text{ ksi}$$

Therefore, the principle tensile stresses at the web just below the top flange is

$$f_{min} = \frac{1}{2} \left( (0.59 + 0) - \sqrt{(0.59 + 0)^2 + [2(0.65)]^2} \right) = -0.42 \text{ ksi}$$

$$f_{min} < \text{Allowable tensile stress of 0.75 ksi} \quad \text{OK}$$
F.1.14. INTERFACE SHEAR TRANSFER

F.1.14.1. Factored Horizontal Shear

At the strength limit state, the horizontal shear at a section on a per unit basis can be taken as:

\[ V_{hi} = \frac{V_u}{d_v} \]  

where

- \( V_{hi} \) = horizontal factored shear force per unit length of the beam, kips/in.
- \( V_u \) = factored shear force at specified section due to superimposed loads after the deck is cast, kips
- \( d_v \) = the distance between the centroid of the tension steel and the mid-thickness of the slab
  
  \[ d_v = (d_e - t_s/2) = 102.50 - 0.0 = 102.50 \text{ in.} \]

It will be assumed here to be the same location as the critical section for vertical shear, at point 0.034 \( L \).

Using load combination Strength I:

\( V_u = 0.0 \text{ kips} \)

Therefore, the applied factored horizontal shear is:

\[ V_{hi} = 0/102.5 = 0.00 \text{ kips/in.} \]

F.1.14.2. Required Nominal Resistance

Required \( V_{hi} = V_{hi}/\phi = 0.9 = 0.00 \text{ kips/in.} \)  

F.1.14.3. Required Interface Shear Reinforcement

The nominal shear resistance of the interface surface is:

\[ V_{ni} = cA_{cv} + \mu[A_{vf}f_{yh} + P_c] \]  

where

- \( c \) = cohesion factor, ksi
- \( A_{cv} \) = area of concrete section resisting shear transfer, in.\(^2\)
- \( \mu \) = coefficient of friction
- \( A_{vf} \) = area of shear reinforcement crossing the shear plane, in.\(^2\)
- \( f_{yh} \) = specified yield strength of shear reinforcement, ksi
- \( P_c \) = permanent net compressive force normal to the shear plane, kips

For cast-in-place conventional concrete slabs placed on clean fluted UHPC girder surface:

\( c = 1.00 \text{ ksi} \)

\( \mu = 1.0 \)

The actual contact width, \( b_v \), between the slab and the beam is 0 in.

\[ A_{cv} = (0)(1.0 \text{ in.}) = 0 \text{ in.}^2 \]

LRFD Eq. 5.7.4.3-3 can be solved for \( A_{vf} \) as follows:

\[ 0 = 1(0) + 1[A_{vf}(60) + 0] \]

Solving for \( A_{vf} \):

\[ A_{vf}(\text{req’d}) = 0 \]

Since the resistance provided by cohesion is greater than the applied force, provide the minimum required interface reinforcement.
LRFD Article 5.7.4.2 states that the minimum reinforcement need not exceed the amount needed to resist $1.33V_{hi}/\Phi$ as determined using Eq. 5.7.4.3.

$1.33(0) = 1(0) + 1[A_v(60) + 0]$

Solving for $A_v$:

$A_v(\text{req'd}) = 0$

**F.1.15. PRETENSIONED ANCHORAGE ZONE**

**F.1.15.1. Anchorage Zone Reinforcement**

Design of the anchorage zone reinforcement is computed using the force in the strands just prior to transfer:

Force in the strands before transfer = $P_{pt} = 54(0.294)(202.5) = 3,214.9$ kips

The bursting resistance, $P_r$, should not be less than 4.0% of $P_{pt}$.

$P_r = 0.50f_{ts}A_{cb} + f_sA_s \geq 0.04P_{pt} = (0.04)(3,215) = 128.6$ kips

where

- $f_{ts}$ = specified first-peak tensile strength for use in design, ksi
- $A_{cb}$ = area of concrete section resisting bursting force within a distance $h/4$ from the end of the beam, in.$^2$
- $A_s$ = total area of vertical reinforcement located with a distance $h/4$ from the end of the beam, in.$^2$
- $f_s$ = allowable stress in steel, but taken not greater than 20 ksi

Solving for the required area of steel, $A_s = \frac{128.6 - 0.50(1)(108/4)(4)}{20} = 3.73$ in.$^2$

At least 3.73 in.$^2$ of vertical transverse reinforcement should be provided within a distance of $(h/4 = 108/4 = 27$ in.) from the end of the beam.

Use ten No. 6 one leg bars at 3-in. spacing starting at 1-in. from the end of the beam.

The provided $A_s = 10(1)(0.44) = 4.40$ in.$^2 > 3.73$ in.$^2$ OK

**F.1.15.2. Confinement Reinforcement**

For a distance of $1.5h = 1.5(108) = 162.00$ in., from the end of the beam, reinforcement is placed to confine the prestressing steel in the bottom flange. The reinforcement may not be less than No. 3 deformed bars with spacing not exceeding 6-in. The reinforcement should be of a shape that will confine (enclose) the strands.

**F.1.16. DEFLECTION AND CAMBER**

Deflections are calculated using the modulus of elasticity of concrete calculated in Section F.1.3.1, and the gross section properties of the noncomposite precast beam.

**F.1.16.1. Elastic Deflection Due to Prestressing Force at Transfer**

$$\Delta_p = \frac{P_{pt}}{E_{ci}I_{cr,i}} \left( \frac{e_{ti}L^2}{8} - e'a^2 \right)$$

where

- $\Delta_p$ = camber due to prestressing force after transfer, in.
- $P_{pt}$ = total prestressing force at transfer = $54(0.294)(202.5) = 3,214.9$ kips
- $e_{ti}$ = eccentricity of prestressing strands at midspan = 50.07 in.
- $e'$ = difference between eccentricity of prestressing strand at midspan and at end of the beam = $e_{ti,mid} - e_{ti,end} = 50.07 - 50.07 = 0$ in.
- $a$ = distance from end of the beam to the harp point = 0 ft
- $L$ = overall beam length = 252 ft
\[ E_{ci} = \text{modulus of elasticity at transfer} = 5,000 \text{ ksi} \]
\[ I_{tr,i} = \text{initial transformed moment of inertia of the noncomposite precast beam} = 2,301,984 \text{ in}^4 \]

Therefore,
\[
\Delta_p = \frac{3.215}{(5,000)(2,301,984)} \left( \frac{(50.07)(252)^2(12)^2}{8} - \frac{(0)(0)^2(12)^2}{6} \right) = 15.99 \text{ in.}
\]

F.1.16.2. Elastic Deflection Due to Beam Self-weight

\[
\Delta_g = -\frac{5w_gL^4}{384E_{ci}I_{tr,i}}
\]
where
\[
\Delta_g = \text{deflection due to beam self-weight, in.}
\]
\[
w_g = \text{beam self-weight} = 1.210 \text{ kips/ft}
\]
\[
L = \text{beam length at transfer} = 252 \text{ ft}
\]
\[
I_{tr,i} = \text{initial transformed moment of inertia of the noncomposite precast beam} = 2,301,984 \text{ in}^4
\]

Therefore,
\[
\Delta_g = -\frac{5(1.21)(252)^4(12)^4}{384(5,000)(2,301,984)(12)} = -9.54 \text{ in.}
\]

F.1.16.3. Elastic Deflection Due to Prestressing Losses (initial to erection)

\[
\Delta_{p,i,e} = -\frac{\Delta P_{pt}}{E_{ci}I_{tr}} \left( \frac{e_{tf}L^2}{8} - \frac{e'^2a^2}{6} \right)
\]
where
\[
\Delta_{p,i,e} = \text{deflection due to prestressing losses between transfer and erection, in.}
\]
\[
\Delta P_{pt} = \text{prestressing force losses between transfer and erection} = 54(0.294)(14.21 + 4.90 + 14.56 + 1.19) = 553.4 \text{ kips}
\]
\[
e_{tf} = \text{eccentricity of prestressing strands at midspan} = 50.95 \text{ in.}
\]
\[
e' = \text{difference between eccentricity of prestressing strand at midspan and at end of the beam} = e_{tf,mid} - e_{tf,end} = 50.95 - 50.95 = 0 \text{ in.}
\]
\[
a = \text{distance from end of the beam to the harp point} = 0 \text{ ft}
\]
\[
L = \text{overall beam length} = 252 \text{ ft}
\]
\[
E_{ci} = \text{modulus of elasticity at transfer} = 6,500 \text{ ksi}
\]
\[
I_{tr,f} = \text{final transformed moment of inertia of the noncomposite precast beam} = 2,248,557 \text{ in}^4
\]

Therefore,
\[
\Delta_{p,i,e} = -\frac{553.4}{(6,500)(2,248,557)} \left( \frac{(50.95)(252)^2(12)^2}{8} - \frac{(0)(0)^2(12)^2}{6} \right) = -2.21 \text{ in.}
\]

F.1.16.4. Deflection Due to Slab and Haunch Weights

\[
\Delta_s = -\frac{5w_sL^4}{384E_{ci}I_{tr,f}}
\]
where
\[
\Delta_s = \text{deflection due to slab and haunch weights, in.}
\]
\[
w_s = \text{slab and haunch weights} = 0 + 0 = 0 \text{ kips/ft}
\]
\[
L = \text{beam length at bearing} = 250 \text{ ft}
\]
\[ I_{tr,f} = \text{final transformed moment of inertia of the noncomposite precast beam} = 2,248,557 \text{ in.}^4 \]

Therefore,
\[ \Delta_s = -\frac{5(0)(250)^4(12)^4}{384(6,500)(2,248,557)(12)} = 0 \text{ in.} \]

**F.1.16.5. Elastic Deflection Due to Barrier and Future Wearing Surface Weights**

\[ \Delta_{b+ws} = -\frac{5(w_b + w_{ws})L^4}{384E_c I_{c,net}} \]

where

- \( \Delta_s \) = deflection due to barrier and wearing surface weights, in.
- \( w_b \) = barrier weight = 0.120 kips/ft
- \( w_{ws} \) = wearing surface weights = 0.208 kips/ft
- \( L \) = beam length at bearing = 250 ft
- \( I_{tr,f} \) = final transformed moment of inertia of the noncomposite precast beam = 2,248,557 in.\(^4\)

Therefore,
\[ \Delta_{b+ws} = -\frac{5(0.12 + 0.208)(250)^4(12)^4}{384(6,500)(2,248,557)(12)} = -1.97 \text{ in.} \]

**F.1.16.6. Elastic Deflection Due to Prestressing Losses (erection to final)**

\[ \Delta_{p\_d\_f} = -\frac{\Delta P_{pt}}{E_c I_{c,net}} \left( \frac{e_t L^2}{8} - \frac{e' \alpha^2}{6} \right) \]

where

- \( \Delta_{p\_d\_f} \) = deflection due to prestressing losses between erection and final, in.
- \( \Delta P_{pt} \) = prestressing force losses between transfer and erection = 54(0.294)(1.31 + 0.68 + 1.19 + 0) = 50.5 kips
- \( e_t \) = eccentricity of prestressing strands at midspan = 50.95 in.
- \( e' \) = difference between eccentricity of prestressing strand at midspan and at end of the beam = \( e_t \)\(_{mid} - e_t \)\(_{end} = 50.95 - 50.95 = 0 \) in.
- \( \alpha \) = distance from end of the beam to the harp point = 0 ft
- \( L \) = overall beam length = 250 ft
- \( E_c \) = modulus of elasticity at transfer = 6,500 ksi
- \( I_{c,net} \) = net transformed moment of inertia of the composite beam = 2,136,129 in.\(^4\)

Therefore,
\[ \Delta_{p\_d\_f} = -\frac{50.5}{(6,500)(2,136,129)} \left( \frac{(50.95)(250)^2(12)^2}{8} - \frac{(0)(0)^2(12)^2}{6} \right) = -0.21 \text{ in.} \]

**F.1.16.7. Deflection Due to Live Load and Impact**

Live load deflection limit (optional) = Span/800

\[ = (250 \times \frac{12}{800}) = 3.75 \text{ in.} \]

If the owner invokes the optional live load deflection criteria specified in LRFD Art. 2.5.2.6.2, the deflection is the greater of:

- that resulting from the design truck plus impact \( \Delta_{LT} \); or

\[ \text{[LRFD Art. 2.5.2.6.2]} \]

\[ \text{[LRFD Art. 3.6.1.3.2]} \]
that resulting from 25% of the design truck plus impact $\Delta_{LT}$ taken together with the design lane load, $\Delta_{LL}$.

Note: LRFD Article 2.5.2.6.2 states that the dynamic load allowance be included in the calculation of live load deflection.

The *LRFD Specifications* states that all the beams should be assumed to deflect equally under the applied live load and impact.

Therefore, the distribution factor for deflection, $DFD$, is calculated as follows:

$$DFD = \frac{\text{number of lanes}}{\text{numbers of beams}}$$

$$= \frac{3}{5} = 0.60 \text{ lanes/beams}$$

**F.1.16.7.1. Deflection due to lane load**

Design lane load, $w = 0.64$ $DFD = 0.64(0.60) = 0.384 \text{ kips/ft/beam}$

$$\Delta_{LL} = -\frac{5wL^4}{384E_cI_c} = -\frac{5(0.384)(250)^4(12)^4}{384(6,500)(2,248,557)(12)} = -2.31 \text{ in.}$$

**F.1.16.7.2. Deflection due to Design Truck Load and Impact**

To obtain maximum moment and deflection at midspan due to the truck load, let the centerline of the beam coincide with the middle point of the distance between the inner 32-kip axle and the resultant of the truck load.

For 8 kips axle, $a = 141.33 \text{ ft}$, $b = 108.67 \text{ ft}$

For 32 kips axle, $a = 127.33 \text{ ft}$, $b = 122.67 \text{ ft}$

For 32 kips axle, $a = 113.33 \text{ ft}$, $b = 136.67 \text{ ft}$

$$\Delta_{LT} = (-2.94)(0.60)(1.33) = -2.35 \text{ in.}$$

Therefore, live load deflection is the greater of:

$$\Delta_{LT} = -2.35 \text{ in.}$$

$$0.25 \Delta_{LT} + \Delta_{LL} = -2.90 \text{ in.}$$

$$\Delta_{LT} \text{ controls at } -2.35 \text{ in.}$$

**F.1.16.8. Camber and Deflection Summary**

To account for long-term effect on the girder’s stiffness, elastic deflection will be multiplied by factor of $(1 + 0.7\psi)$ for losses effect and $(1 + \psi)$ for other loads.
Table F.16-8. Camber and Deflection Summary

<table>
<thead>
<tr>
<th></th>
<th>Deflection Components at Midspan (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Elastic</td>
</tr>
<tr>
<td>Initial prestress</td>
<td>15.99</td>
</tr>
<tr>
<td>Member weight</td>
<td>-9.54</td>
</tr>
<tr>
<td>Loss (initial to erection)</td>
<td>-2.61</td>
</tr>
<tr>
<td>Dead load on composite</td>
<td>-1.97</td>
</tr>
<tr>
<td>Live load</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Deflection Summary at Midspan (in.)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Camber at release</td>
<td>6.45</td>
</tr>
<tr>
<td>Camber at Erection</td>
<td>9.38</td>
</tr>
<tr>
<td>Final (excluding LL)</td>
<td>7.41</td>
</tr>
<tr>
<td>Live load</td>
<td>-2.90</td>
</tr>
<tr>
<td>Span/800</td>
<td>-3.75</td>
</tr>
</tbody>
</table>

The camber at erection is about L/337, which is quite high. It should be noted that camber is a function of the creep coefficient, which is relatively unknown at this time. The creep coefficient will be revised in Phase II. Top prestressing and heat curing will also reduce the predicted camber. Even with a small creep coefficient between release and erection, a camber of 6.45 in. at release, and somewhat higher at erection, may not be acceptable for this type of product where it is expected to be used for accelerated bridge construction (ABC) with only a thin asphalt overlay as the riding surface. In Phase II of the project, the section dimensions will be revised to allow space for the use of top flange unbonded post-tensioning to control camber to a value of about 1 in.

F.1.17. TRANSVERSE DIRECTION DESIGN

The transverse direction design is considered as the design of the ribbed slab, the top flange of the Decked I-Beam. The design procedure is in accordance with the Approximate Elastic Method, Equivalent Strip Method, as described in AASHTO LRFD 4.6.2.1. The ribbed slab is divided into transverse strips supported by rigid support at the center of the beams.
F.1.17.1. Critical Section
AASHTO LRFD specifies a design section for the negative bending moment at the face of the supporting component for stemmed precast beams, cross section Type (j). Thus, the design section is 2” away from the centerline of the beam.

F.1.17.2. Unfactored Dead Loads
The following bending moment calculation for 1 ft wide strip using simplified formula for continuous beams in accordance with AISC Table 3-22c. Conservatively, the bending moment due to dead loads is also taken at the centerline of the beam. The strip should be considered as double cantilevers before curing the CIP joints between the adjacent beams.

Average ribbed slab weight \( w_{slab} = W_{c,tav,deck} (1\text{ ft}) = 0.155(2.875/12)(1) = 0.037 \text{ kips/ft} \)

Weight of closure joint, \( P_{closure} = 0.050 \text{ kips/ft} \)

Weight of barrier, \( w_{barrier} = (2)W_{barrier} (1\text{ ft})/W_{deck} = (2)(0.3)(1\text{ ft})/44.67 = 0.013 \text{ kips/ft} \)

Weight of future wearing surface, \( w_{wss} = W_{sws} (1\text{ ft}) = (0.15)(2/12)(1) = 0.025 \text{ kips/ft} \)

Bending Moment @ Beam Centerline Due to:
1- Ribbed Slab Weight = \( \frac{w_{slab}^2}{2} = \frac{0.037(4.33)^2}{2} = -0.35 \text{ ft-kips/ft} \)
2- Weight of Closure = \( 0.5P_{closure} = -(0.5)(0.05)(4.33) = -0.11 \text{ ft-kips/ft} \)
3- Weight of Barrier = \( 0.107w_{wss}L^2 = -(0.107)(0.013)(9)^2 = -0.11 \text{ ft-kips/ft} \)
4- FWS = \( 0.107w_{barrier}L^2 = -(0.107)(0.025)(9)^2 = -0.22 \text{ ft-kips/ft} \)

Bending Moment @ Mid of the Bay Due to:
1- Ribbed Slab Weight = 0
2- Weight of Closure = 0
3- Weight of Barrier = \( 0.077w_{barrier}L^2 = (0.077)(0.013)(9)^2 = 0.08 \text{ ft-kips/ft} \)
4- FWS = \( 0.077w_{wss}L^2 = (0.077)(0.025)(9)^2 = 0.16 \text{ ft-kips/ft} \)

F.1.17.3. Unfactored Live Loads
The bending moment due to the live load could be determined by modeling a strip of the deck supported by the stems of the beam and applying the two wheels of the truck transversely as a moving load to get the envelope straining actions. These bending moments should be distributed over an equivalent strip width in the longitudinal
direction as specified by AASHTO LRFD 4.6.2.1.3. The moment per one foot with could be calculated by dividing the values from the bending moment diagram by the equivalent strip.

Alternatively, AASHTO LRFD Table A4-1 may be used to calculate the extreme bending moment values due to the truck axle. This tables provide the positive and negative bending moment accounting for the multiple presence factors, the dynamic load allowance, and the equivalent strip.

Maximum Live Loads per unit width @ the Critical Section:
Positive Moment from LL = 6.29 ft-kips/ft
Negative Moment from LL = -6.25 ft-kips/ft

F.1.17.4. Factored Design Moment
Total ultimate bending moment for Strength I is:
\[ M_u = 1.25(\text{DC}) + 1.5(\text{DW}) + 1.75(\text{LL} + \text{IM}) \]
-\[ M_u = 1.25(-0.35 - 0.11 - 0.11) + 1.5(-0.22) + 1.75(-6.25) = -11.98 \text{ ft-kips} \]
+\[ M_u = 1.25(0 + 0 + 0.08) + 1.5(0.16) + 1.75(6.29) = 11.35 \text{ ft-kips} \]

Total ultimate bending moment for Service I is:
\[ M_{SRV} = 1.0(\text{DC}) + 1.0(\text{DW}) + 1.0(\text{LL} + \text{IM}) \]
-\[ M_{SRV} = 1.0(-0.35 - 0.11 - 0.11) + 1.0(-0.22) + 1.0(-6.25) = -7.04 \text{ ft-kips} \]
+\[ M_{SRV} = 1.0(0 + 0 + 0.08) + 1.0(0.16) + 1.0(6.29) = 6.53 \text{ ft-kips} \]

F.1.17.5. Deck Slab Strength Design
The figure below shows a cross section for the ribbed top flange. The width of the design section is 2'-0" to account for one rib with an average width of 3.5". It is considered as Tee-section. For the capacity of the positive moment, the contribution of the fibers may be ignored due to small width of the tension zone. However, the contribution of the fiber is significant for negative moment capacity.

![Figure F.1.17.5-1. Section Through the Top Flange](image)

F.1.17.5.1. Bottom Reinforcement
In order to maintain equilibrium, the sum of the tension and compression forces must equal zero. The process is iterative by assuming the distance from the extreme compression fiber to the neutral axis "c". Assume rectangular section behavior and check if the depth of the equivalent compression stress block, \( a \), is less than or equal to 3":

Note that the iteration has already been done, and only the found value will be shown and used in this section.

\[ c = 0.25 \text{ in.} \]
\[ a = \beta c \]

where
\[ \beta = \text{stress factor of compression block} \]
\[ = 0.85 \text{ for } f'_c \leq 4.0 \text{ ksi} \]
= 0.85 - 0.05(f’c’ - 4.0) ≥ 0.65 for f’c’ > 4.0 ksi

Therefore,
\[ \beta_1 = 0.85 - 0.05(18.0 - 4.0) = 0.15 < 0.65, \text{ use } 0.65 \]
\[ a = (0.65)(0.25) = 0.16 \text{ in.} \]
\[ a < h_f = 2.0 \text{ in.} \]

Therefore, the rectangular section behavior assumption is valid
\[ \varepsilon_s = \varepsilon_c \left( \frac{d_s}{c} - 1 \right) \]

where
\[ \varepsilon_c = \text{ultimate compressive strain} = 0.0035 \]
\[ d_s = \text{distance from extreme compression fiber to the centroid of the prestressing strands} = h - \text{cover} \]
\[ = 8 - 1.5 = 6.5 \text{ in.} \]

Therefore,
\[ \varepsilon_s = 0.0035 \left( \frac{6.5}{0.25} - 1 \right) = 0.075 > 0.004 \]

Check the equilibrium of forces:
\[ \sum F = 0.85 f_c' \beta_1 c b + A_s f_s \]

where
\[ f_c' = \text{specified compressive strength of compression block} = 18.0 \text{ ksi} \]
\[ b = \text{effective width of compression flange} = 24 \text{ in.} \]
\[ A_s = \text{area of nonprestressed tension reinforcement} = 0.6 \text{ in.}^2 \]
\[ f_s = \text{stress in the nonprestressed tension reinforcement} = 100 \text{ ksi} \]

using AASHTO M 334 (ASTM A1035) Grade 100

Sum of compression and tension forces:
\[ \sum F = -0.85(18)(0.65)(24) + (0.6)(100) = 0 \]

Nominal flexural resistance:
\[ M_n = A_s f_s \left( \frac{d_s - a}{2} \right) \]

The above equation is a simplified form of LRFD Equation 5.6.3.2.2-1 because no compression reinforcement is considered and the section behaves as a rectangular section.

\[ M_n = (0.6)(100) \left( 6.5 - \frac{0.16}{2} \right) / 12 = 32.10 \text{ ft-kips} \]

Factored flexural resistance:
\[ M_f = \phi M_n \]

where
\[ \phi = \text{resistance factor} = 0.90, \text{ for tension controlled prestressed concrete sections} \]

Adequate ductility of the beam is ensured by evaluating whether the member can be classified as tension-controlled. If the member does not satisfy the requirements to be tension-controlled, the resistance factor for the strength limit state 1 check will be reduced in accordance with LRFD Article 5.5.4.2.
\[0.75 \leq \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{ct})}{(\varepsilon_{tl} - \varepsilon_{cl})} \leq 0.9\]  
\text{[LRFD Eq. 5.5.4.2-2]}

where

\[\varepsilon_t = \text{net tensile strain in the extreme tension steel at nominal resistance}\]
\[= \varepsilon_s \left(\frac{d_s}{c} - 1\right)\]
\[= 0.003 \left(\frac{6.5}{0.25} - 1\right) = 0.075\]

\[\varepsilon_{cl} = \text{compression-controlled strain limit in the extreme tension steel} = 0.004\]  
\text{[LRFD Art. 5.6.2.1]}

\[\varepsilon_{tl} = \text{tension-controlled strain limit in the extreme tension steel} = 0.008\]  
\text{[LRFD Art. 5.6.2.1]}

Therefore,

\[\phi = 0.75 + \frac{(0.15)(0.075 - 0.004)}{0.008 - 0.004} = 3.41 > 0.9, \text{ use } \phi = 0.90\]

\[M_r = (0.90)(32.10) = 28.89 \text{ ft-kips} > M_u = 22.70 \text{ ft-kips} \quad \text{OK}\]

F.1.17.5.2. Minimum Bottom Reinforcement
At any section, the amount of prestressed and nonprestressed tensile reinforcement must be adequate to develop a factored flexural resistance, \(M_r\), equal to the lesser of:

- 1.33 times the factored moment required by the applicable strength load combination
- The cracking moment determined as follows:

Check at midspan:

\[M_{cr} = \gamma_3 \left[\gamma_1 f_r + \gamma_2 f_{cpw} \right] S_c - M_{dnc} \left(\frac{S_c}{S_{pc}} - 1\right)\]  
\text{[LRFD Eq. 5.6.3.3-1]}

It may be simplified for non-prestressed and non-composite sections as:

\[M_{cr} = \gamma_3 (\gamma_1 f_r) S_c\]

where

\[f_r = \text{specified peak flexural strength, } f_r = 1.5 \text{ ksi}\]
\[S_c = \text{section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads} = 51 \text{ in.}^3\]
\[\gamma_1 = \text{flexural cracking variability factor} = 1.6\]
\[\gamma_3 = \text{ratio of specified minimum yield strength to ultimate tensile strength for AASHTO M 334, Grade 100 reinforcement} = 0.67\]

Therefore,

\[M_{cr} = (0.67)(1.6)(1.5)(51)/12 = 6.83 \text{ ft-kips}\]

At midspan, the factored moment required by the Strength I load combination is:

\[M_u = 22.70 \text{ ft-kips} \quad \text{(from Section F.1.17.4)}\]

Thus, \(1.33 M_u = 1.33(22.70) = 30.19 \text{ ft-kips}\)

Since \(M_{cr} = 6.83 \text{ ft-kips} < 1.33 M_u = 30.19 \text{ ft-kips}\), the \(M_{cr}\) requirement controls

\[M_r = 28.89 \text{ ft-kips} > M_{cr} = 6.83 \text{ ft-kips} \quad \text{OK}\]

F.1.17.5.3. Top Reinforcement
The wide top flange of the ribbed deck provides a significant contribution to the flexural strength capacity, which cannot be ignored. Conservatively, the compression reinforcement may be ignored though. The following calculations show to required values of $M_n$. $M_{n1}$ is the flexural capacity ignoring the fiber contribution. $M_{n2}$ is the flexural capacity accounting for the fiber contribution.

Assuming $A_s = (1)(0.44) = 0.44$ in.$^2$

ASTM A1035, Grade 100 with $f_y = 100.0$ ksi

In order to maintain equilibrium, the sum of the tension and compression forces must equal zero. The process is iterative by assuming the distance from the extreme compression fiber to the neutral axis "c". Assume rectangular section behavior and check if the depth of the equivalent compression stress block, $a$, is less than or equal to 6:

$M_{n1}$ may be determined as follows:

Note that the iteration has already been done, and only the found value will be shown and used in this section.

$c = 1.26$ in.

$a = \beta_1 c$

where

$\beta_1 = \text{stress factor of compression block}$

$= 0.85$ for $f_c' \leq 4.0$ ksi

$= 0.85 - 0.05(f_c' - 4.0) \geq 0.65$ for $f_c' > 4.0$ ksi

Therefore,

$\beta_1 = 0.85 - 0.05(18.0 - 4.0) = 0.15 < 0.65$, use 0.65

$a = (0.65)(1.07) = 0.82$ in.

$a < h_w = 6$ in.

Therefore, the rectangular section behavior assumption is valid

$\varepsilon_s = \varepsilon_c \left( \frac{d_s}{c} - 1 \right)$

where

$\varepsilon_c = \text{ultimate compressive strain} = 0.003$

d_s = \text{distance from extreme compression fiber to the centroid of the prestressing strands} = h - \text{cover}

= 8 - 1.5 = 6.50$ in.

Therefore,

$\varepsilon_s = 0.003 \left( \frac{6.5}{1.26} - 1 \right) = 0.012 > 0.004$

Check the equilibrium of forces:

$\Sigma F = 0.85 f_c' \beta_1 c b + A_s f_s$

where

$f_c' = \text{specified compressive strength of compression block} = 18.0$ ksi

$b_{av} = \text{effective width of compression flange} = 3.50$ in.

$A_s = \text{area of nonprestressed tension reinforcement} = 0.44$ in.$^2$

$f_s = \text{stress in the nonprestressed tension reinforcement} = 100.0$ ksi

using AASHTO M 334 (ASTM A1035) Grade 100

Sum of compression and tension forces:

$\Sigma F = -0.85(18)(0.65)(0.25)(24) + (0.6)(100) = 0$
Appendix F - Design Examples of Bridge Beams

Nominal flexural resistance:
\[ M_n = A_s f_s \left( d_s - \frac{a}{2} \right) \]

The above equation is a simplified form of LRFD Equation 5.6.3.2.2-1 because no compression reinforcement is considered and the section behaves as a rectangular section.

\[ M_{n1} = (0.44)(100) \left( 6.5 - \frac{0.82}{2} \right) / 12 = 22.33 \text{ ft-kips} \]

Factored flexural resistance:
\[ M_r = \phi M_{n1} \]

where
\[ \phi = \text{resistance factor} \]
\[ = 0.90, \text{ for tension controlled prestressed concrete sections} \]

Adequate ductility of the beam is ensured by evaluating whether the member can be classified as tension-controlled. If the member does not satisfy the requirements to be tension-controlled, the resistance factor for the strength limit state 0.9 check will be reduced in accordance with LRFD Article 5.5.4.2.

\[ 0.75 \leq \phi = 0.75 + \frac{0.15(e_t - e_{cl})}{(e_{tl} - e_{cl})} \leq 0.9 \]

where
\[ e_t = \text{net tensile strain in the extreme tension steel at nominal resistance} \]
\[ = \epsilon_s \left( \frac{d_s}{c} - 1 \right) \]
\[ = 0.003 \left( \frac{6.5}{1.26} - 1 \right) = 0.012 \]
\[ e_{cl} = \text{compression-controlled strain limit in the extreme tension steel} = 0.004 \]
\[ e_{tl} = \text{tension-controlled strain limit in the extreme tension steel} = 0.008 \]

Therefore,
\[ \phi = 0.75 + \frac{(0.15)(0.012 - 0.004)}{0.008 - 0.004} = 1.25 > 0.9, \text{ use } \phi = 0.90 \]

\[ M_r = (0.90)(22.33) = 20.10 \text{ ft-kips} \]

\[ M_{n2} \text{ may be determined as follows:} \]
\[ \epsilon_{t,\text{limit}} = \text{the limit of concrete tensile strain} = 0.004 \]

From Curvature Moment Diagram:
\[ \epsilon_c = \text{ultimate compressive strain} = 0.0023 \]
\[ \epsilon_c \text{ is less than } 0.85 f'_c / E_c. \text{ Therefore, the compression stress is distributed as a triangle.} \]

By iteration,
\[ c = 2.92 \text{ in.} \]
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

Figure F.1.17.5.3-1. Strain and Strain Distributions Over the Depth of the Ribbed Slab

\[ \varepsilon_s = \varepsilon_c \left( \frac{d_s}{c} - 1 \right) \]

where

\[ d_s = \text{distance from extreme compression fiber to the centroid of the prestressing strands} = h - \text{cover} = 8.00 - 1.50 = 6.5 \text{ in.} \]

Therefore,

\[ \varepsilon_s = 0.0023 \left( \frac{6.5}{2.99} - 1 \right) = 0.003 < 0.004 \]

Check the equilibrium of forces:

\[ \sum F = 0.5 \varepsilon_c E_c b_{av} + A_s f_s + 0.50 f_{\text{limit}} X_1 b + f_{\text{limit}} X_2 b + f_{\text{limit}} X_3 B \]

where

\[ f_{\text{c}}' = \text{specified compressive strength of compression block} = 18.0 \text{ ksi} \]
\[ b_{av} = \text{effective width of compression flange} = 3.50 \text{ in.} \]
\[ A_s = \text{area of nonprestressed tension reinforcement} = 0.44 \text{ in.}^2 \]
\[ f_s = \text{stress in the nonprestressed tension reinforcement} = 71.92 \text{ ksi} \]
\[ f_{\text{limit}} = \text{the cracking tensile strength limit of the idealized plateau} = 0.75 \text{ ksi} \]
\[ X_1 = \text{the depth of the triangle in the tension zone} \]
\[ X_2 = \text{the depth of the rectangle in the tension zone within the web, in.} \]
\[ X_3 = \text{the depth of the rectangle in the tension zone within the flange, in.} \]
\[ B = \text{the flange width} = 24.0 \text{ in.} \]

To calculate \( X_2 \) and \( X_3 \), find the strain at the intersection between the web and the flange. Additionally, the tensile strain limit should be located.

\[ \varepsilon_{t,2} = \text{strain at the intersection between the web and the flange} \]
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

\[
\frac{h_w - c}{\varepsilon_c} = \frac{(6 - 2.92)\cdot 0.0023}{2.92} = 0.0024
\]

\[
X_2 = \frac{h_w - X_1 - c}{\varepsilon_c} \quad \text{if } \varepsilon_{t,2} \leq \varepsilon_{t,\text{limit}}
\]
\[
= \frac{(\varepsilon_{t,\text{limit}})}{\varepsilon_c} c \quad \text{if } \varepsilon_{t,2} > \varepsilon_{t,\text{limit}}
\]

\[
= 6 - 0.15 - 2.92 = 2.93 \text{ in.}
\]

\[
\varepsilon_{t,3} = \text{strain at the extreme tension fibers}
\]

\[
= \frac{(h - c)\varepsilon_c}{c} = \frac{(8 - 2.92)\cdot 0.0023}{2.92} = 0.004
\]

\[
X_3 = h - X_1 - X_2 - c \quad \text{if } \varepsilon_{t,3} \leq \varepsilon_{t,\text{limit}}
\]
\[
= \frac{(\varepsilon_{t,\text{limit}})}{\varepsilon_c} c - X_1 - X_2 \quad \text{if } \varepsilon_{t,3} > \varepsilon_{t,\text{limit}}
\]

\[
= 8 - 0.15 - 2.92 = 2.00 \text{ in.}
\]

Sum of compression and tension forces:

\[
\Sigma F = -0.5(0.0023)(6500)(2.92)(3.5) + (0.44)(71.92) + 0.5(0.75)(0.15)(3.5) + (0.75)(2.93)(3.5) + (0.75)(2)(24) = 0
\]

Nominal flexural resistance:

\[
M_n = A_s f_s \left( d_s - \frac{c}{3} \right) + 0.5 f_{\text{limit}} X_1 b \left( h - \frac{c}{3} - \left( X_3 + X_2 + \frac{X_1}{3} \right) \right) + f_{\text{limit}} X_2 b \left( h - \frac{c}{3} - \left( X_3 + \frac{X_2}{2} \right) \right)
\]

\[
+ f_{\text{limit}} X_3 b \left( h - \frac{c}{3} - \frac{X_3}{2} \right)
\]

The above equation is a modified form of LRFD Equation 5.6.3.2.2-1 with no compression reinforcement considered, and the section behaves as a rectangular section.

\[
M_n = \left[ (0.44)(71.92)\left( 6.5 - \frac{2.92}{3} \right) + 0.50(0.75)(0.15)(3.5)\left( 8 - \frac{2.92}{2} - \left( 2 + 2.93 + \frac{0.15}{3} \right) \right) \right. \\
\left. + (0.75)(2.93)(3.5)\left( 8 - \frac{2.92}{3} - \left( 2 + \frac{2.93}{2} \right) \right) + (0.75)(2)(24) \left( 8 - \frac{2.92}{3} - \frac{2}{2} \right) \right]/12
\]

\[
M_n = 34.97 \text{ ft-kips}
\]

Factored flexural resistance:

\[
M_{r,2} = \phi M_n\]

where

\[
\phi = \text{resistance factor} \quad [\text{LRFD Art. 5.5.4.2}]
\]

\[
= 0.75, \text{ for compression controlled non-prestressed concrete sections}
\]

Adequate ductility of the beam is ensured by evaluating whether the member can be classified as tension-controlled. If the member does not satisfy the requirements to be tension-controlled, the resistance factor for the strength limit state 0.9 check will be reduced in accordance with LRFD Article 5.5.4.2.

\[
0.75 \leq \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{t,1} - \varepsilon_{cl})} \leq 0.9 \quad [\text{LRFD Eq. 5.5.4.2-2}]
\]

where

\[
\varepsilon_t = \text{net tensile strain in the extreme tension steel at nominal resistance}
\]
\[
= \varepsilon_s\left( \frac{d_s}{c} - 1 \right)
\]
\[ \varepsilon_{cl} = 0.0023 \left( \frac{6.5}{2.99} - 1 \right) = 0.003 \]

- Compression-controlled strain limit in the extreme tension steel = 0.004 [LRFD Art. 5.6.2.1]

\[ \varepsilon_{tl} = 0.008 \]

- Tension-controlled strain limit in the extreme tension steel = 0.008 [LRFD Art. 5.6.2.1]

Therefore,
\[ \phi = 0.75 + \frac{(0.15)(0.003 - 0.004)}{0.008 - 0.004} = 0.71 \]

\[ = 0.71 < 0.75, \text{ use } \phi = 0.75 \]

\[ M_{r2} = (0.75)(34.97) = 26.23 \text{ ft-kips} \]

To ensure ductility, \( M_{nl} \) should not be less than 0.8\( M_u \).

\[ M_{nl} = 22.33 \text{ ft-kips} > F.17 \text{ ft-kips} = 0.8(2 \times 11.98) = 0.8 M_u \]

\[ \phi M_u = 26.23 \text{ ft-kips} > M_u = (2)(11.98) = 23.96 \text{ ft-kips} \]

OK

**F.1.17.5.4. Minimum Top Reinforcement**

At any section, the amount of prestressed and nonprestressed tensile reinforcement must be adequate to develop a factored flexural resistance, \( M_r \), equal to the lesser of:

- 1.33 times the factored moment required by the applicable strength load combination
- The cracking moment determined as follows:

Check at midspan:

\[ M_{cr} = \gamma_3 \left( \gamma_1 f_r + \gamma_2 f_{cp} \right) S_c - M_{nc} \left( \frac{S_c}{S_{nc}} - 1 \right) \]  

[LRFD Eq. 5.6.3.3-1]

It may be simplified for non-prestressed and non-composite sections as:

\[ M_{cr} = \gamma_3 (\gamma_1 f_r) S_c \]

where

- \( f_r \) = specified peak flexural strength, \( f_r = 1.5 \text{ ksi} \)
- \( S_c \) = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads = 136 in.\(^3\)
- \( \gamma_1 \) = flexural cracking variability factor = 1.6
- \( \gamma_3 \) = ratio of specified minimum yield strength to ultimate tensile strength for AASHTO M 31, Grade 60 reinforcement = 0.67

Therefore,
\[ M_{cr} = (0.67)(1.6)(1.5)(136)/12 = 18.22 \text{ ft-kips} \]

At midspan, the factored moment required by the Strength I load combination is:

\[ M_u = 23.96 \text{ ft-kips} \text{ (from Section F.1.17.4)} \]

Thus, 1.33\( M_u = 1.33(23.96) = 31.87 \text{ ft-kips} \)

Since \( M_{cr} = 18.22 \text{ ft-kips} < 1.33 M_u = 31.87 \text{ ft-kips} \), the \( M_{cr} \) requirement controls

\[ M_r = 27.11 \text{ ft-kips} > M_{cr} = 18.22 \text{ ft-kips} \]

OK

**F.1.17.5.5. Overhang Reinforcement**

Bridge deck overhangs shall be designed for the three separate design cases: [AASHTO LRFD A13.4.1]

- Design Case 1: Horizontal and longitudinal forces from vehicle collision load (Extreme Event II limit state)
- Design Case 2: Vertical force from vehicle collision load (Extreme Event II limit state)
- Design Case 3: Vertical Dead and Live Load at the overhang section (Strength I limit state)

Design Case 1 more than likely controls the design of the overhang. Thus, this case will only be considered in this example. The applied loads are vehicular collision plus the dead load. In this case, the vehicular collision introduces axial tension force on the deck. The deck shall be able to handle the collision forces and have higher resistance capacity than the barrier. The barrier should be designed first to provide some required parameters, which is out of the scope of this example. Therefore, the barrier parameters will be assumed as shown below.

It also should be noted that two sections should be designed. First is at the toe of the barrier and second is at the critical section near to the web of the beam. Conservatively, only the critical section will be considered with neglecting any increase of the critical length of yield line away from the barrier.

\[ X_{\text{barrier}} = \text{width of barrier base} = 18.00 \text{ in.} \]
\[ X_{\text{cg}} = \text{barrier center of gravity from deck edge} = 6.50 \text{ in.} \]
\[ T = \text{axial load per unit length} = 7.50 \text{ kip/ft} \]
\[ M_c = \text{moment capacity of the barrier} = 16.50 \text{ kips-ft/ft} \]
\[ L_c = \text{critical length of yield line} = 10 \text{ ft} \]
\[ W_{\text{overhang}} = \text{overhang width} = 4.33 \text{ ft} \]
\[ X_{\text{DS}} = \text{design section to girder CL} = 2.00 \text{ in.} \]

**Bending Moment @ Critical Section Due to:**

1. Ribbed Slab Weight = \(0.5\cdot w_{\text{slab}}W_{\text{overhang}} = -0.5(0.037)(4.33)^2 = -0.35 \text{ ft-kips/ft} \)
2. Weight of Barrier = \(P_{\text{barrier}}(X_{\text{overhang}} - X_{\text{DS}} - X_{\text{cg}}) = -0.3(4.33 - 2/12 - 6.5/12) = -1.09 \text{ ft-kips/ft} \)
3. FWS = \(0.5\cdot w_{\text{overhang}}(X_{\text{overhang}} - X_{\text{DS}} - X_{\text{DS}}) = -0.5(0.025)(4.33 - 2/12 - 18/12)^2 = -0.09 \text{ ft-kips/ft} \)
4. Collision = \(M_c = -16.5 \text{ ft-kips/ft} \)

**Design Factored Moment (Extreme Event II)**

\[ M_u = 1.0\cdot M_{\text{DE}} + 1.0\cdot M_{\text{DW}} + 1.0\cdot M_c = 1.0(-0.61 + -1.09) + 1.0(-0.09) + 1.0(-16.5) = 18.03 \text{ ft-kips/ft} \]

**Design Factored Axial Tension Force (Extreme Event II)**

\[ T_u = 1.0\cdot T = 7.5 \text{ kips/ft} \]

Factored flexural resistance for 2 ft wide strip:

\[ M_{n1,\text{overhang}} = M_{nt} - T_{\text{axial}} \left( \frac{h}{2} - \frac{a}{2} \right) = 22.33 - 2(7.5) \left( \frac{8}{2} - \frac{0.82}{2} \right) / 12 = 17.84 \text{ ft-kips} \]
\[ M_{n2,\text{overhang}} = M_{n2} - T_{\text{axial}} \left( \frac{h}{2} - \frac{a}{2} \right) = 36.14 - 2(7.5) \left( \frac{8}{2} - \frac{0.82}{2} \right) / 12 = 31.14 \text{ ft-kips} \]

To ensure ductility, \( M_{n1} \) should not be less than \( 0.8M_u \)

\[ M_{n1} = 17.84 \text{ ft-kips} < 28.80 \text{ ft-kips} = 0.8(2 \times 18.03) = 0.8M_u \]

If not, \( \phi M_n \) should be taken as the larger of \( \phi M_{n1} \) or \( 0.67 M_{n2} \)

\[ \phi M_n = \max[0.9(17.84), 0.67(31.14)] = 20.86 \text{ ft-kips} < M_u = (2)(18.29) = 36.58 \text{ ft-kips} \]

\[ \text{NG} \]

**F.1.18. PUNCHING SHEAR**

The top skin of the deck between ribs needs to be checked against punching shear of the wheel patch load. The skin should sustain HS20 truck (16 kips per tire) and Tandem truck (12.5 kips per tire). The tire contact area of a wheel shall be assumed a single rectangle of 20-in. by 10-in. (LxW) (LRFD 3.6.1.2.5).

**F.1.18.1. Factored Punching Shear**

At the strength limit state, the critical punching shear force of HS20 wheel can be taken as:

\[ V_u = 1.75P(AI)m \]

where

- \( V_u \) = factored punching shear force due to wheel load, kips
- \( P \) = concentrated wheel load = 16.0 kips
- \( I_g \) = dynamic allowance = 1.33
- \( m \) = multiple presence factor = 1.2

Therefore,

\[ V_u = 1.75(16)(1.33)(1.2) = 44.7 \text{ kips} \]

**F.1.18.2. Required Nominal Resistance**

Required: \( V_n = V_u / \phi = 44.7 / 0.9 = 49.7 \text{ kips} \)

**F.1.18.3. Provided Nominal Resistance**

The nominal punching shear resistance of the top skin is:

\[ V_n = 0.125\sqrt{f'_c} b_o h_f \]  

[LRFD Eq. 5.8.4.3.4-3]

where

- \( h_f \) = depth of deck top skin = 2.00 in.
- \( b_o \) = the perimeter of the critical section for shear enclosing the tire contact area, in.
  - \( = 2L + 2W + 2h_f \)
  - \( = 2(20) + 2(10) + 2(2) = 64.00 \text{ in.} \)

Therefore,

\[ V_n = 0.125\sqrt{18.0(64)(3)} = 67.9 \text{ kips} > 49.7 \text{ kips} \]

Therefore, the deck top skin can sustain the wheel load

**F.1.19. STABILITY ANALYSIS**

The wide top flange of a 250-ft Decked I-beam may lead to a risk of rolling over during transportation and/or at case of single girder on bearing. These cases should be carefully addressed in addition to all other cases. Figure F.1.19-1 shows the eccentricities of the girder, used to calculate the stability values. Girder Stability Analysis V1.0 was utilized to evaluate the safety against cracking and overturning of the decked I-beam. The rotational spring constant of the shipping support and center-to-center wheel spacing were assumed 80,000 kip-in/rad and 96 in., respectively.
The effective prestressing needs to be calculated for the cases of when lifting from the bed and when being transported, and can be seen as follows:

Effective prestressing at release and lifting from the bed = $f_p - \Delta f_{pES} - \Delta f_{pSHI} = 202.5 - 20.63 - 14.18 = 167.69$ ksi

Therefore, the total prestressing force when lifting from the bed = $(167.69)(15.88) = 2,662.9$ kips

Effective prestressing at transportation = $f_p - \Delta f_{pES} - \Delta f_{pSHI} - \Delta f_{pLT} = 202.5 - 20.63 - 14.18 - 14.8 = 152.89$ ksi

Therefore, the total prestressing force at transportation = $(152.89)(15.88) = 2,427.9$ kips

F.1.19.1. Lifting with Vertical or Inclined Cables

Parameters of Being Lifted with Vertical or Inclined Cables:

- $y_{lift}$ = lift connection rigid extension above top of girder = 9.00 in.
- $\varepsilon_{conn}$ = lift connection lateral tolerance from centerline of beam = 0.50 in.
- $w_{wind,Lift1}$ = lateral wind pressure at lifting from bed = 0.028 klf
- $w_{wind,Lift2}$ = lateral wind pressure at lifting in field = 0.028 klf

<table>
<thead>
<tr>
<th>Stage</th>
<th>$f'_c$</th>
<th>$P_{eff}$</th>
<th>$\varepsilon_{cs,mid}$</th>
<th>$\varepsilon_{total}$</th>
<th>Camber</th>
<th>$\alpha$</th>
<th>IM</th>
<th>$FS_{cr}$</th>
<th>$FS'$</th>
<th>$f_b,eq$</th>
<th>$f_c,eq$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lifting from Bed</td>
<td>10.0</td>
<td>2663</td>
<td>4.37</td>
<td>3.15</td>
<td>6.40</td>
<td>20.0</td>
<td>20%</td>
<td>1.77</td>
<td>1.77</td>
<td>3.936</td>
<td>0.626</td>
</tr>
<tr>
<td>Lifting in Field</td>
<td>18.0</td>
<td>2427</td>
<td>4.37</td>
<td>4.15</td>
<td>7.27</td>
<td>20.0</td>
<td>20%</td>
<td>1.99</td>
<td>1.99</td>
<td>3.396</td>
<td>0.654</td>
</tr>
</tbody>
</table>

Parameters of Being Lifted with Inclined Cables:

- $y_{upper,yolk}$ = height of upper yolk above lower yolk = 184.0 ft
Implementation of UHPC in Long-Span Precast Pretensioned Elements

Appendix F - Design Examples of Bridge Beams

**Figure F.1.19.1-2. Free Body Diagram of the Girder Being Lifted with Inclined Cables**

**Table F.1.19.1-2. Input and Results of Girder Stability when Lifting with Inclined Cables**

<table>
<thead>
<tr>
<th>Stage</th>
<th>$f'_c$</th>
<th>$P_{eff}$</th>
<th>$y_{cgs,mid}$</th>
<th>$e_{total}$</th>
<th>Camber</th>
<th>$a$ (ft)</th>
<th>IM (%</th>
<th>$FS_{cr}$</th>
<th>$FS'$</th>
<th>$f_{b,eq}$</th>
<th>$f_{t,eq}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lifting from Bed</td>
<td>10.0</td>
<td>2662</td>
<td>4.37</td>
<td>3.15</td>
<td>6.40</td>
<td>20</td>
<td>20%</td>
<td>1.78</td>
<td>1.78</td>
<td>3.904</td>
<td>0.626</td>
</tr>
<tr>
<td>Lifting in Field</td>
<td>18.0</td>
<td>2427</td>
<td>4.37</td>
<td>4.15</td>
<td>7.27</td>
<td>20</td>
<td>20%</td>
<td>2.02</td>
<td>2.02</td>
<td>3.367</td>
<td>0.654</td>
</tr>
</tbody>
</table>

**F.1.19.2. Girder Seated on Dunnage**

**Figure F.1.19.2-1. Free Body Diagram of the Girder Seated on Dunnage**

Parameters of Seating on Dunnage:
- $y_{seat1}$ = height from roll center to beam seat = 2.00 in.
- $e_{seat1}$ = bunking tolerance from CL beam to CL support = 0.00 in.
- $K_{seat1}$ = dunnage foundation stiffness = 200000 kip-in/rad
- $\alpha_{seat1}$ = maximum seating tolerance from level = 0.010 ft/ft
- $W_{wind,seat1}$ = lateral wind pressure at seating on dunnage = 0.028 klf
Table F.1.19.2-1. Input and Results of Girder Stability when Seated on Dunnage

<table>
<thead>
<tr>
<th>Stage</th>
<th>( f'c )</th>
<th>( P_{eff} )</th>
<th>( y_{cgs,mid} )</th>
<th>( \varepsilon_{\text{total}} )</th>
<th>Camber</th>
<th>( a )</th>
<th>( F_{S_{cr}} )</th>
<th>( F_{S'} )</th>
<th>( F_{S_{roll}} )</th>
<th>( f_{b,eq} )</th>
<th>( f_{t,eq} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dunnage</td>
<td>10.0</td>
<td>2662</td>
<td>4.37</td>
<td>3.15</td>
<td>6.40</td>
<td>2.0</td>
<td>4.67</td>
<td>4.67</td>
<td>2.58</td>
<td>3.479</td>
<td>0.967</td>
</tr>
</tbody>
</table>

F.1.19.3. Girder Seated on Transportation

![Free Body Diagram of the Girder Seated on Transportation](image)

Parameters of Seating on Transportation:
- \( \varepsilon_{\text{bunk,trans}} \) = bunking tolerance from CL beam to CL support = 0.50 in.
- \( K_{\text{trans}} \) = hauling rig stiffness = 80,000 kip-in/rad
- \( \alpha_{\text{trans}} \) = maximum superelevation = 0.020 ft/ft
- \( \text{Radius}_{\text{trans}} \) = maximum turn radius = -120.0 ft
- \( V_{\text{e,trans}} \) = design speed in turn = 4.0 mph
- \( y_{\text{seat,trans}} \) = height from roll center to beam seat = 48.00 in.
- \( z_{\text{max,trans}} \) = horizontal distance from roll axis to center of tire group = 48.00 in.
- \( h_{\text{roll,trans}} \) = height of roll center above roadway = 24.00 in.
- \( w_{\text{wind,trans}} \) = lateral wind pressure at seating on transport = 0.028 klf

Table F.1.19.3-1. Input and Results of Girder Stability when Seated on Transportation

<table>
<thead>
<tr>
<th>Stage</th>
<th>( f'c )</th>
<th>( P_{eff} )</th>
<th>( y_{cgs,mid} )</th>
<th>( \varepsilon_{\text{total}} )</th>
<th>Camber</th>
<th>( a )</th>
<th>IM</th>
<th>( F_{S_{cr}} )</th>
<th>( F_{S'} )</th>
<th>( F_{S_{roll}} )</th>
<th>( f_{b,eq} )</th>
<th>( f_{t,eq} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transport</td>
<td>18.0</td>
<td>2427</td>
<td>4.37</td>
<td>4.15</td>
<td>7.27</td>
<td>36.0</td>
<td>10%</td>
<td>1.47</td>
<td>4.67</td>
<td>1.62</td>
<td>4.555</td>
<td>-0.416</td>
</tr>
</tbody>
</table>
F.1.19.4. Single Girder on Bearing

Parameters of Single Girder on Bearing:

\( y_{\text{seat.brg2}} \) = height from roll center to beam seat = 2.00 in.
\( e_{\text{brg.seat2}} \) = bearing tolerance from CL beam to CL support = 0.50 in.
\( K_{\text{eseat2}} \) = bearing rotational stiffness = 74,725 kip-in/deg.
\( a_{\text{seat2}} \) = equivalent bunk points = 1.0 ft
\( \alpha_{\text{seat2}} \) = maximum transverse seating tolerance from level = 0.005 ft/ft
\( w_{\text{wind.seat2}} \) = lateral wind pressure at seating on bearing of first girder = 0.028 klf

<table>
<thead>
<tr>
<th>Stage</th>
<th>( f_c )</th>
<th>( P_{\text{eff}} )</th>
<th>( y_{\text{cgs.mid}} )</th>
<th>( e_{\text{I,total}} )</th>
<th>Camber</th>
<th>a</th>
<th>FS(_{\text{cr}})</th>
<th>FS'</th>
<th>FS(_{\text{roll}})</th>
<th>( f_{\text{b.eq}} )</th>
<th>( f_{\text{Eq}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>On Bearing</td>
<td>18.0</td>
<td>2427</td>
<td>4.37</td>
<td>4.15</td>
<td>7.27</td>
<td>1.0</td>
<td>1.69</td>
<td>1.64</td>
<td>0.54</td>
<td>3.024</td>
<td>0.674</td>
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<tr>
<td>Check with No End Bracing</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>N.G., Add End Bracing</td>
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</tr>
</tbody>
</table>

F.1.19.5. Girder Seated at Inactive Construction

Parameters of Multiple Seated Girders at Inactive Construction:

\( w_{\text{wind.seat3}} \) = lateral wind pressure at seating on bearing, inactive construction = 0.135 klf
\( w_{\text{lift.seat3}} \) = lateral wind uplift at seating on bearing, inactive construction = 0.013 klf
\( n_{\text{brace}} \) = number of brace, including at end of girder \( \geq 2 \)
Imperfection (play) in each brace = 0.25 in.
### Table F.1.19.5-1. Parameters and Results of Girder Stability at Inactive Construction

<table>
<thead>
<tr>
<th>Stage</th>
<th>$f'_c$</th>
<th>$P_{eff}$</th>
<th>$y_{cgs,mid}$</th>
<th>$e_{total}$</th>
<th>Camber</th>
<th>$a$</th>
<th>$FS_{cr}$</th>
<th>$FS'$</th>
<th>$f_{b,eq}$</th>
<th>$f_{i,eq}$</th>
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<tr>
<td>No Bracing Check</td>
<td>18.0</td>
<td>2427</td>
<td>4.37</td>
<td>4.15</td>
<td>7.27</td>
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<td>0.77</td>
<td>1.61</td>
<td>3.803</td>
<td>-1.393</td>
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<td>Check w/ No Intermediate Bracing</td>
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<td></td>
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<tr>
<td>Bracing Check</td>
<td>Check with Intermediate Bracing</td>
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<td>-0.002</td>
<td>Bracing Adequate</td>
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