ERDC/GSL TR-23-16

Geotechnical and Structures Laboratory

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Navigation Systems Research Program

Development and Characterization of Ultra-High-Performance Concrete for the Rehabilitation of Navigation Lock Structures

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Final report

Prepared for Navigation Systems Research Program (NavSys)
US Army Corps of Engineers
Washington, DC   20314-1000

Under Project 493064, “UHPC Panels”

Abstract

This report details the history of vertical lock wall repairs and the development and laboratory characterization of an ultra-high-performance concrete (UHPC) using locally sourced materials for improved durability of lock walls subjected to impact and abrasion from navigational vessels. This UHPC, referred to as Lock-Tuf, has been designed for use in a precast environment with ambient curing methods and serves as a material proof-of-concept for future lock wall rehabilitations. Mechanical properties such as unconfined compressive strength, flexural response, tensile capacity, impact resistance, and abrasion resistance have been quantified experimentally.

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Preface

This study was conducted for the US Army Corps of Engineers' Navigation Systems Research Program (NavSys) under Project 493064, "UHPC Panels."

The work was performed by the Concrete and Materials Branch (GM-C) of the Engineering Systems and Materials Division (GM), US Army Engineer Research and Development Center, Geotechnical and Structures Laboratory (ERDC-GSL). At the time of publication, Dr. Jameson D. Shannon was chief, GM-C; Mr. Justin S. Strickler was chief, GM; and Mr. Charles E. Wiggins was the technical director for Navigation. The deputy director of ERDC-GSL was Mr. Charles W. Ertle II, and the director was Mr. Bartley P. Durst.

COL Christian Patterson was the commander of ERDC, and Dr. David W. Pittman was the director.

1 Introduction

1.1 Background

This project was developed based on Statement of Need submission 2018-N-1244, entitled “Ultra High-Performance Concrete for the Repair of Navigation and Other Marine Structures Subject to Impact and Abrasion,” submitted by the US Army Corps of Engineers (USACE) Rock Island District (MVR). The project was prioritized for funding under the Engineer Research and Development Center (ERDC) Navigation Research Area Review Group (RARG).

Many USACE concrete navigation structures are currently functioning beyond their designed service lives and suffer from damage and deterioration, particularly along guide walls and lock chamber wall surfaces. External damage from vessel contact and internal concrete deterioration mechanisms, such as freeze–thaw cycling, often develop a synergetic relationship, compounding the deleterious effects on concrete. Efforts have been made to improve lock wall durability. Cast-in-place concrete for new construction and repairs began including the embedment of steel armor in the lock wall surface to protect against vessel impact and abrasion as well as air entrainment in the concrete to reduce cracking caused by freeze–thaw cycling. An inspection conducted on several Ohio River locks nearly four decades after armored cast-in-place concretes were put into service found significant damage to both the steel armor and the surrounding concrete (Lewis et al. 2011). The inspection report detailed instances of localized areas of broken armor and gouges and spalls in the concrete adjacent to steel armor strips.

The use of cast-in-place concrete for lock wall repair presents additional drawbacks. Because formwork must first be installed prior to casting the concrete, repair projects require complex construction logistics with increased costs associated with materials and labor, and locks may experience relatively long operation downtimes. New concrete that is cast against and bonded with older concrete also tends to exhibit cracking caused by restraint combined with phenomena associated with curing (i.e., moisture loss, thermal cooling, and hydration reactions). Efforts to overcome restraint cracking—such as utilization of bond breakers, variations in materials, and optimization of mixture proportions—were largely unsuccessful (McDonald 1987). The resulting cracks permit moisture ingress, further contributing to concrete deterioration.

A precast concrete stay-in-place forming system was developed as part of the USACE Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program that took place in the late 1980s and early 1990s at ERDC, then called Waterways Experiment Station (WES) (ABAM Engineers Inc. 1987). The use of precast panels eliminated the restraint cracking issue and the need for formwork, resulting in a more durable repair that requires less lock operation downtime. The design, which uses normal strength concrete and steel armor, is still in use today. Panels are typically 6 to 8 in.* thick and heavy, requiring large cranes on site that increase construction costs and interfere with lock activities. The amount of required steel reinforcement, susceptibility of normal strength concrete to deterioration mechanisms, and potential corrosion of the steel armor contribute to enduring durability concerns.

Ultra-high-performance concrete (UHPC) is a class of cementitious materials with specified compressive strength of at least 22,000 psi, water-to-cementitious materials ratio (w/cm) of less than 0.25, and improved ductility and durability compared to normal strength concrete (Perry and Zakariasen 2004; ACI Committee 239 2018). To achieve these characteristics, UHPCs are made with specific high-quality materials including, but not limited to, oil well or low-heat portland cement (i.e., large mean particle size, high C_2S content and low C_3A content), siliceous or aluminous fine aggregates, crushed quartz or some other micrometer-sized powder, silica fume, high-range water-reducing admixtures to control rheology, and other components that vary by manufacturer. Because concrete tends to exhibit more brittle behavior as compressive strength is increased, steel fibers are included in UHPC to bind and delocalize micro- and macro-cracking, resulting in improved tensile strength.

UHPC is a viable material alternative for lock wall repair and rehabilitation. Its high strength and density contribute to superior

* For a full list of the spelled-out forms of the units of measure used in this document, please refer to *US Government Publishing Office Style Manual*, 31st ed. (Washington, DC: US Government Publishing Office, 2016), 248–252, https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf.

resistance to impact and abrasion as well as moisture ingress and transport that drive common concrete deterioration mechanisms. UHPC panels can be made significantly thinner than ordinary concrete panels, will eliminate the need for external steel armoring, and will reduce the amount of required steel reinforcement, thereby simplifying design, installation, and maintenance requirements. Although UHPC typically costs more than normal strength concrete, it is expected that introducing precast UHPC panels as a repair material will ultimately result in long-term cost savings to USACE Districts.

1.2 Scope

The overall scope of this project was to evaluate the potential application of UHPC precast panels in the repair of deteriorated lock walls. To do so, an understanding of current and historical lock wall rehabilitations was required as well as the evolution of construction practices and materials as they relate to field performance. Five key factors were identified for this mission: documentation of historical lock wall rehabilitations, UHPC development using local materials, laboratory performance, structural performance, and recommendations for inclusion of UHPC in Engineering Manuals. This report outlines three of these factors: historical rehabilitations, UHPC development, and laboratory performance. The remaining two factors are to be published in separate reports. The recommendations for the inclusion of UHPC in Engineering Manual 110-2-2000 can be found in an ERDC Special Report, ERDC/GSL SR-23-1 *Suggested Updates for the Inclusion of Guidance on Ultra-High-Performance Concrete to USACE Engineering Manual 1110-2-2000, Standard Practice for Concrete for Civil Works Structures* (2023).

1.3 Objectives

The objectives of this project were to (1) develop a nonproprietary UHPC mixture based on particle packing density, using the same materials typically used by precast concrete manufacturers, (2) demonstrate the ability to successfully cast UHPC large-scale test samples at a precast manufacturing plant, where the labor force may be unfamiliar with the material, using the plant's processes, facilities, and skillsets, (3) evaluate the impact and abrasion resistance of the nonproprietary UHPC in small- and large-scale tests, (4) use project outcomes to inform needed updates on UHPC development, testing, and applications in USACE Engineer Manuals (EMs), and (5) perform a cost benefit analysis to determine the potential cost savings to USACE Districts over time using UHPC panels instead of the armored, normal strength concrete panels.

This report contains details related to objectives (1) and (3). A brief history of lock wall repair and rehabilitation methods is included as select case studies in MVR. The development of the nonproprietary UHPC is described, and small-scale laboratory testing and results of the nonproprietary UHPC compared to a normal strength concrete and a proprietary UHPC are discussed. Large-scale casting and testing will be captured in a future ERDC technical report (TR). Performance specifications for the development of UHPC mixtures for use in precast panels for this application will also be provided in a future ERDC TR.

1.4 Approach

Within this manuscript, a detailed history of lock wall rehabilitations within the Rock Island District is provided, showcasing the historical evolution of the rehabilitation practices. UHPC material development with local materials sourced from a nearby precast plant is detailed as proof-of-concept for material development and constructability of this class of concrete with existing commercial infrastructure. Laboratory-scale characterizations of normal-strength concrete, proprietary UHPC, and a nonproprietary, ERDC-developed UHPC are provided to illustrate the improved performance of UHPC over conventional concrete and that UHPCs can be developed and fielded using local materials, lowering the financial burden associated with proprietary blends.

2 US Army Corps of Engineers (USACE) Lock Wall Repair History

The USACE operates and maintains more than 100 locks located in areas of relatively severe exposure to freeze–thaw cycling. Because many of these locks were built before 1940, air entrainment was not used in the mixtures, and many of these structures exhibit significant concrete deterioration. The extent of deterioration ranges from surface scaling to several feet in depth. The deterioration on the chamber walls generally extends down to just below the low water surface.

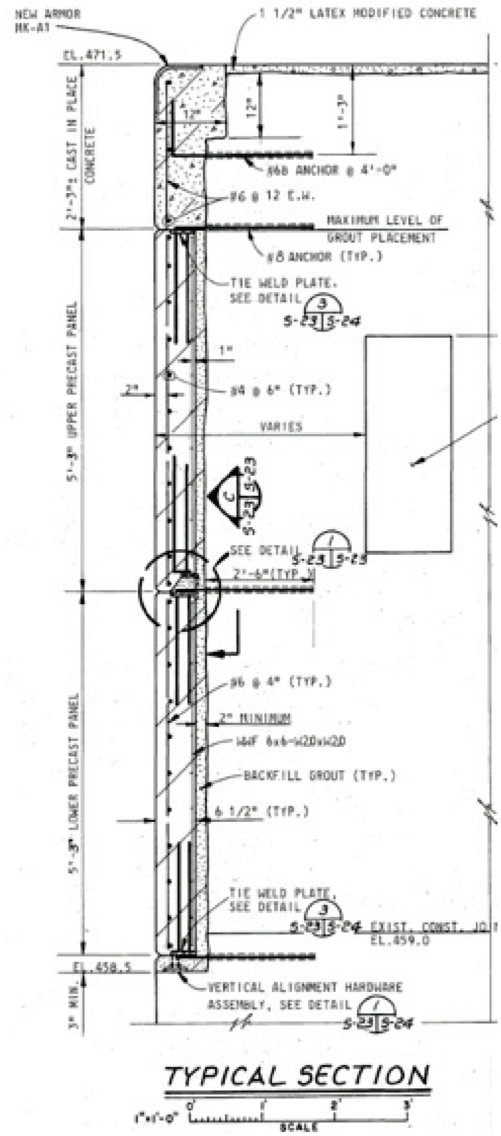
The following sections present lock wall repair and rehabilitation case studies in MVR to illustrate the evolution of repair materials and methods.

2.1 Lock 22—Precast Panel Repair

In 1989, the walls of Lock 22 were resurfaced with precast concrete panels. The concrete removal area extended 13 ft down from the top of the lock wall and the entire length of the chamber between the miter gates. The depth of concrete removal was nominally 8.5 in. and was accomplished by line drilling and blasting. The precast concrete panels were 6.5 in. thick and were 5 ft 3 in. high with lengths that varied from 19 to 37 ft, depending on monolith joint spacing, with the majority of panels being 30 ft long. The panels were reinforced with 6 × 6-W20 × W20 welded wire fabric on the inner face and #4 horizontal rebar spaced at 6 in. on-center (o.c.) and #6 vertical rebar spaced at 4 in. o.c., as shown in Figure 1. Other embedded steel items included weld plates on 2 in. centers at the top and bottom of the panel, leveling inserts on 5 in. centers along the bottom of the panel, and stripping and erection handles. There was no embedded steel armor in the panels.

The panels were secured to the existing wall by welding #8 rebar, which was epoxied into the existing wall to the weld plates at 2 in. o.c. at the top and bottom of the panel, as shown in Figure 1. The precast concrete panel mixture was designed to be air entrained and have a 28-day compressive strength of 6,500 psi. Material used to infill the 2 in. space between the precast panel and the existing wall was a grout mix with a 28-day strength of 4,000 psi. The upper 2 ft. 6 in. of the wall was cast-in-place (CIP) concrete with the steel corner armor embedded in the placement.

Figure 1. Lock 22—Typical section of precast panel used in repair.



After the lock reopened to navigation, the vertical precast panel monolith joints began to exhibit spalling from abrasion damage caused by barges rubbing against the wall, which exhibited a slight misalignment of the panels combined with untapered joints. To remedy the spalling, the joints were tapered to a 1 in. depth over a 1 ft length on each side of the joint, as shown in Figure 2. The panels have been in service for 30 yr and are generally in good condition with the exception of the joint damage.

Figure 2. Lock 22 precast panels in February 2018.

2.2 Lock 13—Cast-in-Place Repair

In the winter of 1995, the lock walls of Lock 13 were resurfaced with CIP concrete. The concrete removal area extended 22 ft down from the top of the lock wall and the entire length of the chamber between the miter gates. The depth of concrete removal was nominally 14 in. and was done by saw cutting, line drilling, and blasting with final cleanup by jackhammers and excavator-mounted milling machines. The CIP concrete was reinforced with #6 rebar spaced at 12 in. o.c. each way. The new concrete was anchored to the existing concrete with drilled and epoxied #6 rebar anchors at 4 in. o.c. each way, as shown in Figure 3. Wall and top corner armor was secured to the forms before concrete placement. The CIP concrete mixture was an air-entrained conventional 4,000 psi mixture with 1.5 in. nominal maximum size aggregate.

Restraint cracking due to shrinkage developed in the new CIP concrete. The cracking has not significantly contributed to deterioration of the concrete after 24 yr of service, as shown in Figure 4. The closure period lasted approximately 90 days. Significant time was taken to fabricate the forms and attached the armor to the forms for each set.

Figure 3. Lock 13—Typical lock wall section for cast-in-place (CIP) concrete repair.

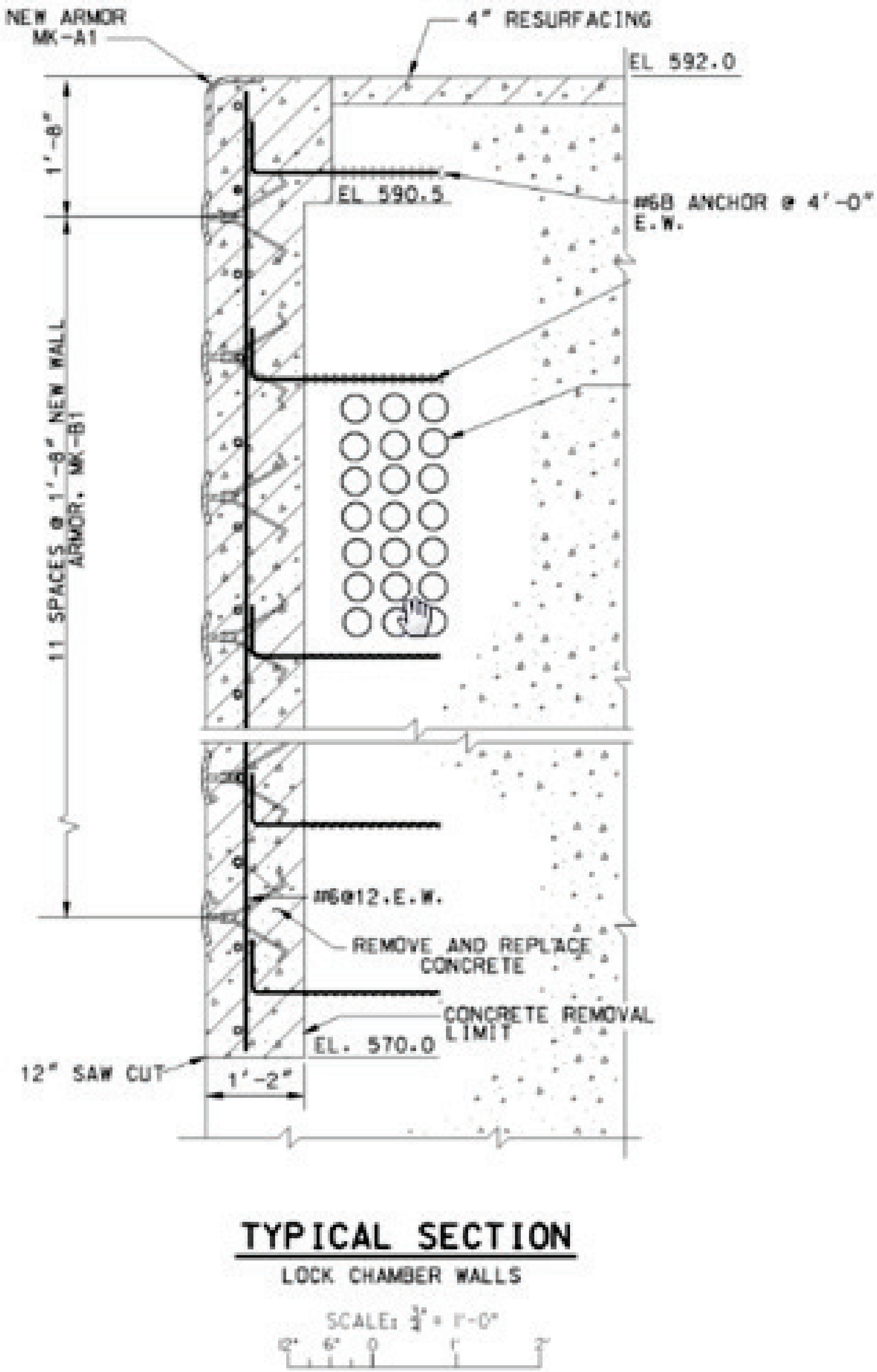


Figure 4. Lock 13—Cast-in-place concrete repair in winter 2016.

2.3 Lockport—Cast-in-Place Repair

In the summer of 1995, the lock walls of Lockport Lock were repaired with CIP. The concrete removal area extended 22 ft down from the top of the lock wall and the entire length of the chamber between the miter gates. The depth of concrete removal was nominally 14 in. and was done by saw cutting, line drilling, and blasting. The CIP concrete was reinforced with #6 rebar spaced at 12 in. o.c. both ways, as shown Figure 5. The new concrete was anchored to the existing concrete by drilled and epoxied #6 bent anchors spaced at 4 in. o.c. both ways. Top corner armor and wall armor were secured to the forms before concrete placement. Due to the height of the repair, wall armor was included only in the upper and lower portions of the wall where abrasion from tows would occur. Figure 6 shows construction after milling and at the beginning of formwork installation. The repair required dewatering of the lock chamber and several pieces of heavy equipment. The CIP concrete was an air-entrained conventional 4,000 psi mixture with 1.5 in. nominal maximum size aggregate. The lock closure period lasted 140 days with the contractor working 24 hr a day 7 days a week. Restraint cracking occurred and was noticeable around the armor, as shown in Figure 7. The cracking has not significantly contributed to deterioration of the concrete after 24 yr of service, as shown in Figure 8.

Figure 5. Lockport Lock—Typical repair section using CIP concrete.

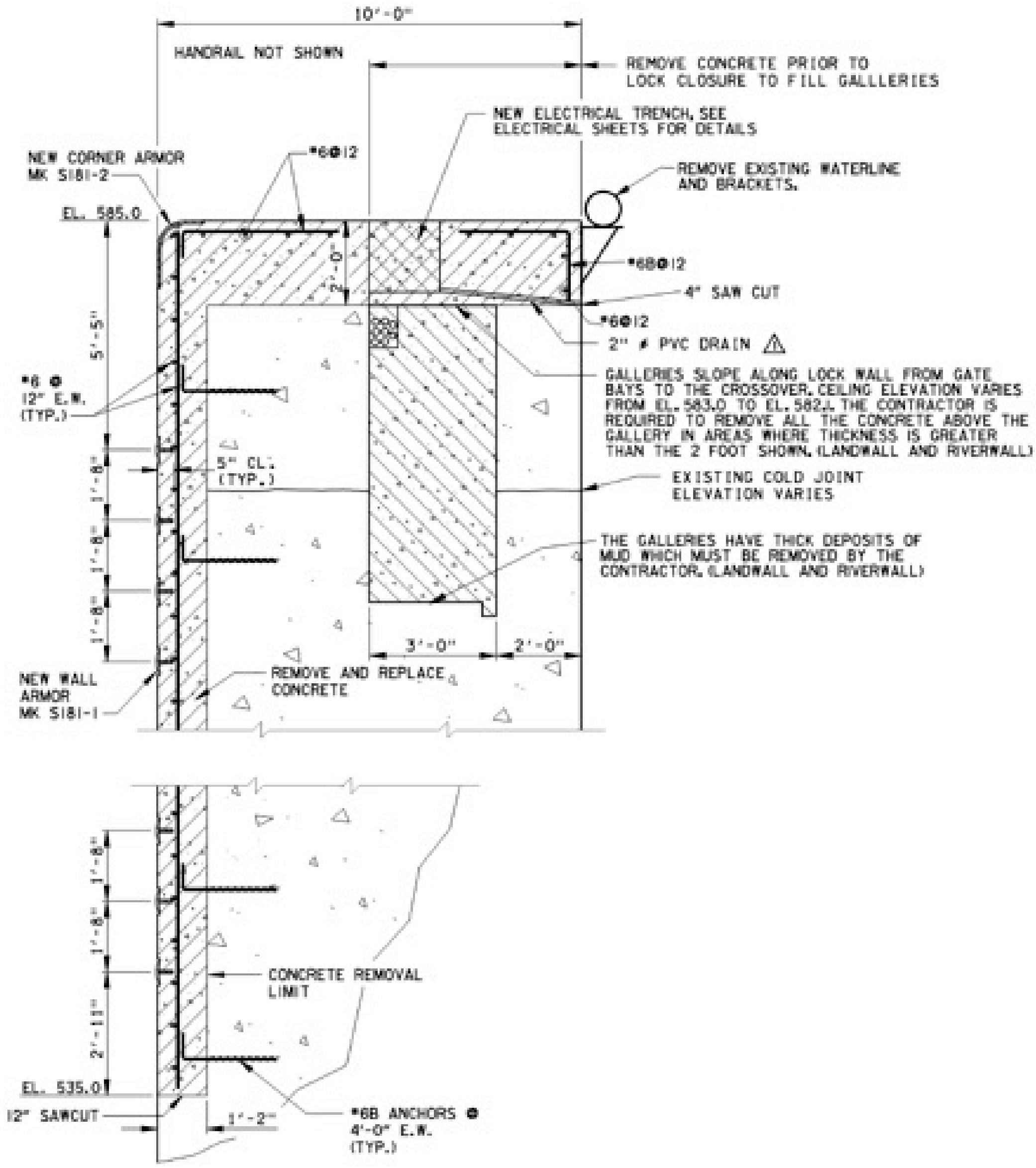


Figure 6. Lockport Lock wall during repair.

Figure 7. Lockport—Typical restraint cracking in CIP repair concrete.

Figure 8. Lockport Lock CIP repair concrete in 2010.

2.4 Lock 12 and Lock 11—Precast Panel Repair

Lock 12 and Lock 11 walls were repaired 2001–2002 and 2006–2007, respectively, with the new generation of precast panels using tapered joints. The concrete removal area extended 21.5 ft at Lock 12 and 23.5 ft at Lock 11 down from the top of the lock wall and the entire length of the chamber between the miter gates. The depth of concrete removal was nominally 12 in. and was accomplished by line drilling and blasting.

The precast concrete panel heights were 10 ft 7.25 in. and 10 ft 9.75 in. for the upper and lower panels, respectively, at Lock 12 and 11 ft 7.5 in. and 10 ft 9.5 in. for the upper and lower panels, respectively, at Lock 11. The panel lengths varied from 10 to 31 ft 10.5 in., depending on the monolith joint spacing. The 6.5 in. thick panels were reinforced with epoxy coated #6 vertical bars spaced 12 in. o.c. and #6 horizontal bars spaced 18 in. o.c. and had hooked bars protruding out the back side to anchor to the wall. The panels included embedded wall armor and top corner armor, as shown in Figure 9. The edges of the panel were tapered 0.75 in. over 5.5 in. on Lock 12 and 1 in. over 6 in. on Lock 11 to prevent spalling caused by abrasion. The panels were secured to the existing wall by drilled and epoxied #6 bent embedded anchors and a #5 bar threaded through the panel anchors and the #6 bent anchors. The concrete in the precast panel was an air-entrained mixture with a 6,000 psi strength at 28 days, utilizing 5% silica fume to increase durability and reduce permeability.

The infill concrete that was placed between the precast panel and the existing concrete was an air-entrained mixture with a 4,000 psi strength at 28 days. The panels were erected by installing the first row of panels on shims to level the panels at the correct elevation. The panels were temporarily supported during construction with whalers mounted on the existing lock wall to support the bottom of the panel and angle clips on top of the panel, which were bolted to anchors epoxied into the existing concrete. After the lower row of panels was leveled and aligned, the infill concrete was placed in lifts not exceeding 4 ft to limit pressure on the precast panel stay-in-place forms. The lifts were spaced out to assure at least one row of anchors was engaged within each lift. The infill concrete was placed along the entire length of the wall with no joints installed at the monolith joints. Because work occurred during cold weather, the panels were insulated and heated to allow for proper curing of the infill concrete. After the lower row of panels was installed and the infill concrete was placed, the upper row of panels was erected in a similar manner.

At Lock 12, there was minor spalling along the joint between the upper and lower panels. This issue was corrected in the Lock 11 project by grouting the gap between the upper and lower panels, as shown in Figure 10. The closure periods for Lock 12 and Lock 11 were greatly shortened by utilizing the precast panels. The time taken from setting the first panel to finishing infill concrete behind the last panel at Lock 12 was only 21 days. Postlock wall rehabilitation pictures of Lock 12 and Lock 11 are shown in Figures 11 and 12, respectively.

Figure 9. Lock 11—Typical section of repair with precast panels.

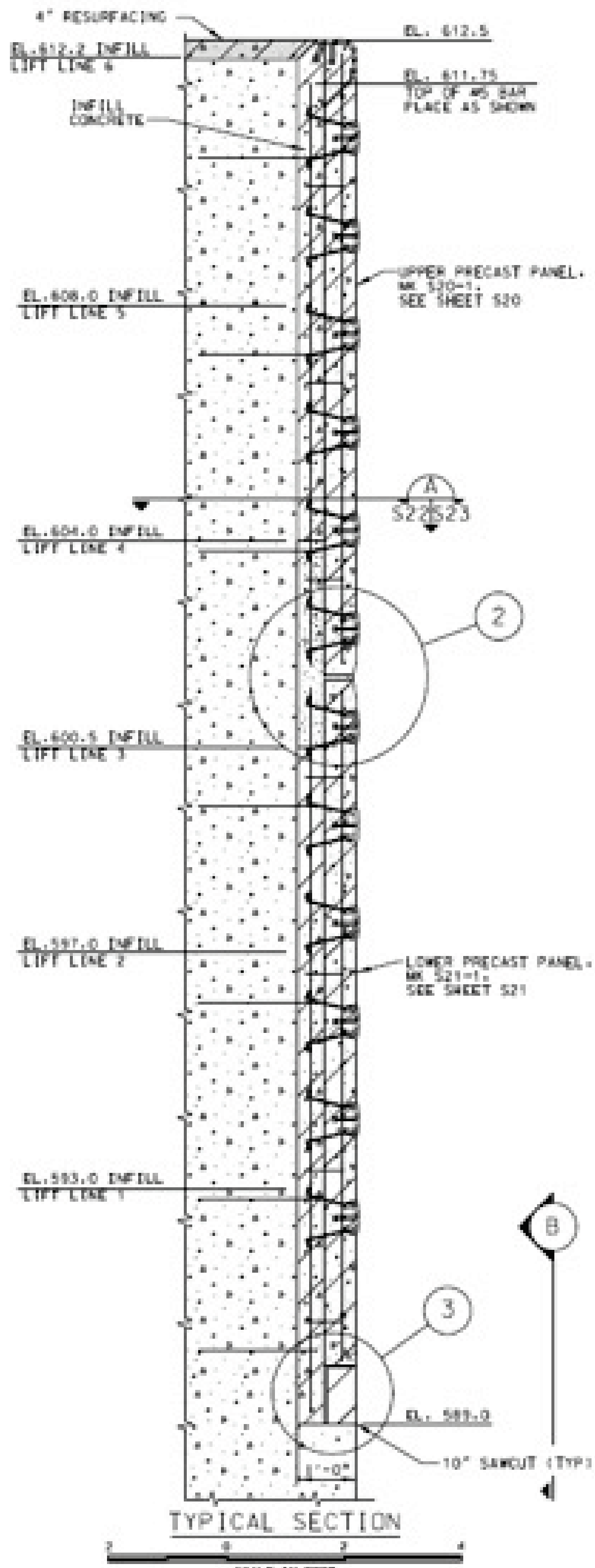


Figure 10. Lock 11—Upper and lower panel joint detail.

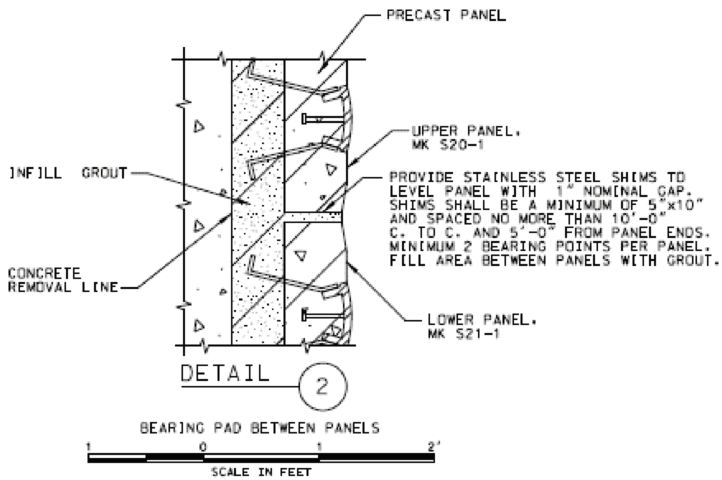


Figure 11. Lock 12—Precast panel repair in 2014.

Figure 12. Lock 11—Precast panel repair in 2014.

2.5 Summary of Repair History

Repair and rehabilitation case studies are summarized in Table 1, which includes the repair method, repair year(s), surface concrete specifications, infill or backfill, concrete specifications, closure period, and notes on the current conditions of the repairs.

Table 1. Summary of lock wall repair case studies with notes on current repair conditions.

Lock #	Method	Year	Concrete Specs.	Infill Specs.	Closure Period (Days)	Current Condition Notes
22	Precast	1989	6,500 psi @ 28 days; Air-entrained;	Grout – 4,000 psi @ 28 days	—	Vertical joint spalling
13	CIP	1995	4,000 psi @ 28 days; Air-entrained; 1.5-in. NMSA	n/a	~90	Restraint / shrinkage cracking shortly after placement

Lock #	Method	Year	Concrete Specs.	Infill Specs.	Closure Period (Days)	Current Condition Notes
Lockport	CIP	1995	4,000 psi @ 28 days; Air-entrained; 1.5-in. NMSA	n/a	140	Restraint / shrinkage cracking shortly after placement
12	Precast	2001–2002	6,000 psi @ 28 days; Air-entrained; 5% silica fume	4,000 psi @ 28 days; Air-entrained	21	Minor spalling along joints between upper and lower panels.
11	Precast	2006–2007	6,000 psi @ 28 days; Air-entrained; 5% silica fume	4,000 psi @ 28 days; Air-entrained	—	Grout used to prevent spalling between upper and lower panels.

Precast concrete panels have evolved into an efficient method of repairing lock walls by minimizing field formwork, reducing placement of concrete reinforcement in the field, reducing lock closure durations, eliminating restraint cracking of the outer surface, and producing higher quality concrete by casting the panels in a controlled environment. Further improvements may be realized by using UHPC rather than normal strength concrete. Should the impact and abrasion resistance of UHPC be sufficient for this application, panels made with UHPC would provide protection against both damage and deterioration while improving casting and installation logistics.

3 Concrete Materials

This study compared an ERDC-developed, nonproprietary UHPC mixture to normal strength concrete and a proprietary UHPC using small-scale test methods, including those for impact and abrasion resistance. The normal strength concrete chosen represented materials and proportions similar to what is typically used in the current precast panel design.

3.1 Normal Strength Concrete (PAC-5)

The normal strength concrete chosen for this research was an ERDC laboratory standard 5,000 psi concrete mixture known as PAC-5. The design 28-day compressive strength of 5,000 psi was selected as a mean between the 4,000 to 6,500 psi concrete strengths specified in previous lock wall rehabilitations.

PAC-5, formerly known as SAC-5, was originally designed to be a normal strength concrete that used materials readily available in most of the United States. The materials used to produce PAC-5 include a 3/8 in. chert, or pea gravel, from Crystal Springs, Mississippi; Green Brothers concrete sand from Crystal Springs; an ASTM C595-21 portland limestone cement (2021a); an ASTM C618-19 Type C fly ash (2019a); and water-reducing admixtures from BASF Master Builders.

3.2 Proprietary Ultra-High-Performance Concrete (UHPC) (Ductal)

Ductal is a family of proprietary UHPCs developed and sold by LafargeHolcim. It is marketed as a UHPC with unmatched qualities of durability, aesthetics, and strength. Its major applications are for bridges, roads, and architectural projects. Ductal is sold with all dry materials preblended into either 50 lb paper sacks or two different sizes of super sacks. The fibers are Bekaert Dramix OL 13/.20 mm fibers and can be procured with the purchase of Ductal. The admixture is a high-range water-reducing one known as Chryso. This is a highly refined and readily available UHPC material. However, it is very expensive, uses nonlocal materials, and requires specialized mixing equipment.

For this study, Ductal J1000 was used as a standard UHPC due to its wide availability and past use with many projects across the United States, mainly in bridge deck and joint construction. J1000 has an unconfined compressive strength of about 30,000 psi after steam curing. The steam curing begins 24 hr after placement and continues for three days at 190 deg Fahrenheit.

3.3 Nonproprietary UHPC (Lock-Tuf)

The ERDC-developed, nonproprietary UHPC was given the name Lock-Tuf. It was conceptualized and developed as a proof-of-concept UHPC that would provide improved impact and abrasion resistance versus normal strength concrete but would do so at a reduced cost and logistical burden compared to Ductal and other proprietary UHPCs. It was developed during a series of trial batches using local materials sourced from Tindall Corporation precast plant in Moss Point, Mississippi. It was also designed to be producible in an existing rotating pan mixer at Tindall that is similar to equipment found at most precast plants across the country. Tindall in Moss Point was chosen because of its locality to ERDC and its existing knowledge base developed during multiple military engineering (ME) programs and partnerships with Tindall Corp.

Twelve trial mixtures were leveraged from ME programs and tailored for civil works (CW) applications and production in a precast environment. From these 12 mixtures, four were downselected to be cast as part of this research. All constituent materials were sourced from the Tindall Corporation precast concrete plant in Moss Point, except for the silica fume and the steel fibers. The available concrete materials included Type III cement, concrete sand, Class F fly ash, a high-range water-reducing admixture (HRWRA), a slump retention admixture, silica fume, and OL 13/.20 mm steel fibers. All technical data sheets are presented in Appendix B. The final mixture was selected based on fresh and hardened properties.

During the scope development of this project, the sole US-based manufacturing plant for the OL 13/.20 fibers was shut down, generating concerns about the US government’s ability to procure them under the 1933 Buy American Act. Therefore, alternative fiber types were selected for use in Lock-Tuf. However, another OL fiber manufacturing plant quickly began production, and it was decided that the alternative fiber mixtures would nevertheless be tested to evaluate their performance in comparison with the original Lock-Tuf using OL fibers. All fiber volume fractions were held at a constant 2% for all Lock-Tuf mixtures. Three alternative mixtures were made. The first mixture incorporated a larger, hooked-end steel fiber known as 3D 55/30 BG from Bekaert. The second was a basalt fiber. The third alternative mixture utilized a multiscale fiber approach with a combination of a steel wool microfiber, as described in Ragalwar et al. (2020), and the 3D 55/30 BG macrofiber.

4 Experimental Methods

The experimental methods described in this section were followed to evaluate the mechanical performance of PAC-5, Lock-Tuf, and Ductal. In addition, the alternative mixtures of Lock-Tuf using different fiber types were characterized to identify an optimal fiber type.

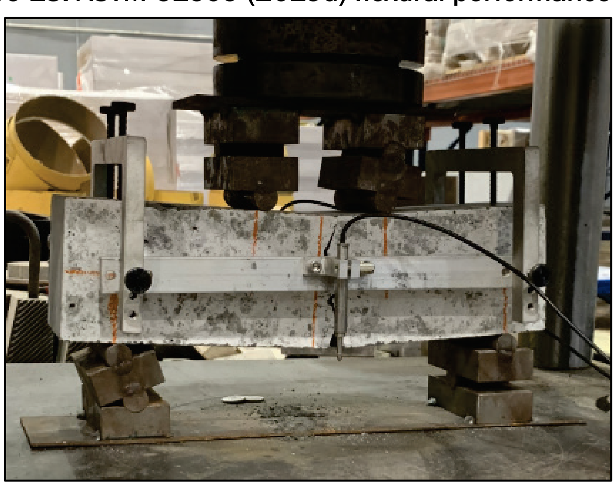
4.1 Unconfined Compressive Strength

The unconfined compressive strength was determined in accordance with ASTM C39-21, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens* (2021b). Compressive strength testing was conducted during trial mixture proportioning and mixture characterization. Due to trial batch size limitations, freeze–thaw, abrasion, and impact, test specimens were cast as separate batches. Compressive strength was also taken on specimens cast for those test methods for quality assurance.

4.2 Flexural Performance

The flexural performance testing was performed according to ASTM C1609-19, *Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)* (2019d) and is shown in Figure 13. The beams were 6 in. × 6 in. × 21 in. and tested using an 18 in. span. The beams were cast according to ASTM C192-19, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory* (ASTM International 2016). Linear variable differential transformers (LVDTs) were used to measure displacement, which was paired with the corresponding load data to provide a plot of load versus displacement.

Figure 13. ASTM C1609 (2019d) flexural performance test.



4.3 Freeze–Thaw Cycling

Many locks within the USACE area of responsibility are located in cold winter climates and are subjected to many freeze–thaw cycles over time. UHPC has been well-documented for its superior performance in freeze–thaw conditions compared to normal strength concrete. This improved performance is largely due to a reduced permeability and increased tensile capacity of the concrete due to fiber reinforcement (Graybeal and Tanesi 2007; Tanesi et al. 2004). However, a proposed reduction in lock wall panel thickness using UHPC required verification that reduced thicknesses and reinforcement cover depths would not cause deterioration under freeze–thaw conditions.

Because of the reduced cover depth in the UHPC panels and the lingering concerns of corrosion, MVR requested large-scale testing of an unreinforced panel and a panel reinforced with glass-fiber-reinforced polymer (GFRP) bars in lieu of conventional steel. There is a greater difference between the coefficients of thermal expansion of UHPC and GRFP than there is between UHPC and steel, generating concerns over potential debonding of the GFRP from the UHPC after cycles of freezing and thawing. To evaluate this behavior, reinforced beams using either conventional steel or GFRP were tested first in ASTM C666 (2015) then subjected to flexural testing in accordance with ASTM C1609 (2019d).

4.3.1 Unreinforced Beams

Freezing and thawing durability testing was accomplished in accordance to Procedure A of ASTM C666-15, *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing* (2015). The wet, or rapid, freeze–thaw experiment is shown in Figure 14. Concrete prisms with dimensions of 3 in. × 4 in. × 16 in. were moist-cured for 14 days before being subjected to freeze–thaw cycles until failure or a maximum of 300 cycles. The specimens were cycled between 4 and -18°C in 2 hr, so that 12 freezing and thawing cycles were executed in 24 hr. Fundamental transverse frequency was measured approximately every 36 cycles. Results are reported as the durability factor, which is a function of the number of cycles survived by the specimens and the relative dynamic modulus of elasticity at the time the test was terminated. The relative dynamic modulus testing is shown in Figure 15.

Figure 14. ASTM C666 (2015) freeze–thaw testing.

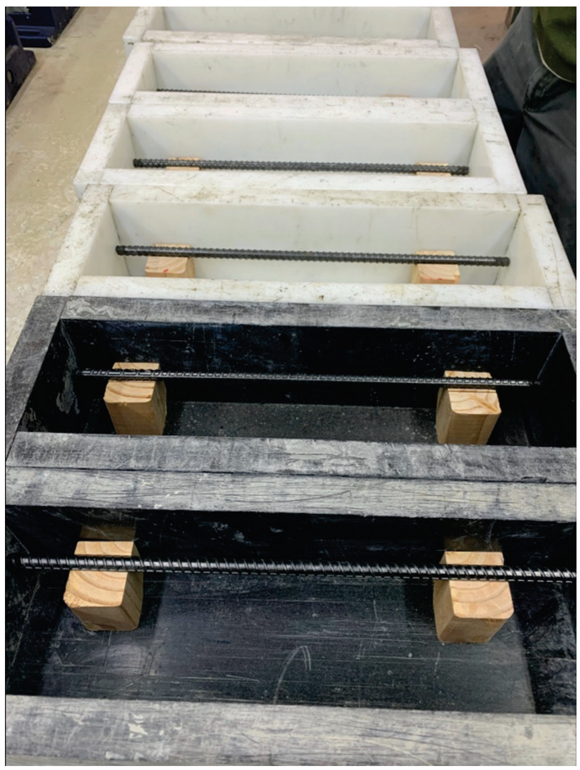
Figure 15. Relative dynamic modulus testing of a freeze–thaw beam.

4.3.2 Reinforced Beams

Size #3 GFRP and steel bars were cast in 4 in. × 4 in. × 18 in. flexure beams with a clear cover depth of ½ in. from the side face of each beam. The specimen size was chosen so that the beams would fit into the freeze–thaw chamber containers and could then be tested in the three-point bending test. The rebar was positioned so that it would be centered along the tensile face of the beam during flexural loading. The reinforcement embedment procedure is shown in Figure 16. Wooden blocks were cut to size and used to support the rebar while each end was epoxied to the end faces of the beam molds. Once the epoxy cured, the wooden blocks were removed prior to casting.

Six flexural performance freeze–thaw beams were cast for each reinforcement type (GFRP and steel) using Lock-Tuf. Three beams for each type of reinforcement were subjected to 300 cycles of freezing and thawing in accordance with ASTM C666 (2015) while the remaining three beams were kept as a control set in a moist curing room at 100% relative humidity. All 12 beams were then tested for flexural performance according to ASTM C1609 (2019d) within a few days after the conditioned beams had reached the 300-cycle target.

Figure 16. Epoxying steel rebar in reinforced beams tested in ASTM C666 (2015) and ASTM C1609 (2019d).



4.4 Direct Tension

Direct tension testing was conducted according to a method developed by the Federal Highway Administration (FHWA) (Graybeal and Baby 2013). UHPC can exhibit improved tensile mechanical properties, including sustained postcracking tensile strength, compared to normal strength concrete. Direct tension methods are better able to capture the linear elastic, strain-hardening, and strain-softening behavior of UHPC than indirect test methods such as splitting-tensile strength (Graybeal and Baby 2013).

ERDC partnered with FHWA's Turner-Fairbank Highway Research Center in McClean, Virginia, to perform direct tension tests on Lock-Tuf and Ductal samples. The test method utilized a 2 in. × 2 in. × 17 in. prism with epoxied metal grip plates forming a 4 in. center gauge length with four extensometers mounted along the central faces across the gauge length section (Graybeal and Baby 2013). Figure 17 shows the prisms being clamped while the grip plates are being epoxied to the sample. Graybeal and Baby (2013) provides detailed pictures of the experimental setup. Testing of the UHPC specimens is shown in Figure 18. A constant displacement rate of 0.0001 in./s was used.

Figure 17. Epoxied grip plates on prism samples for direct tension testing.

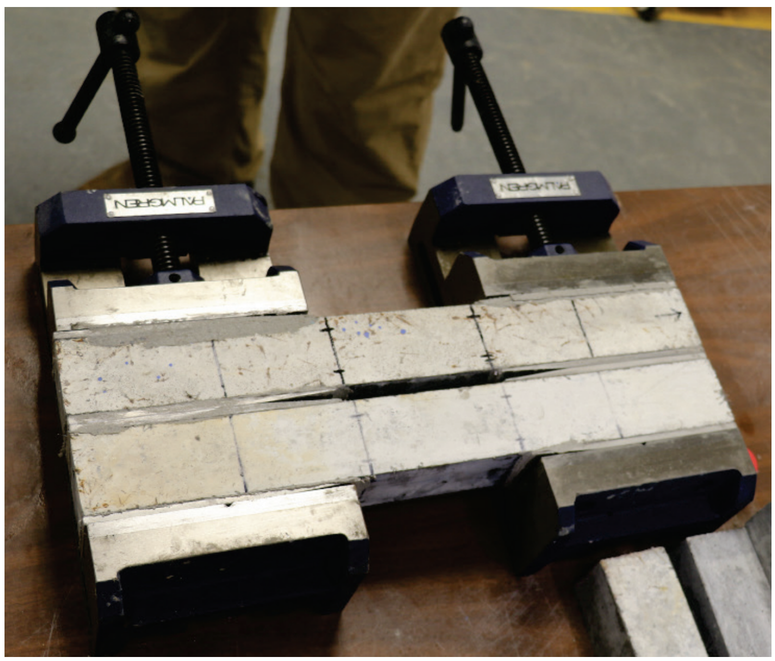
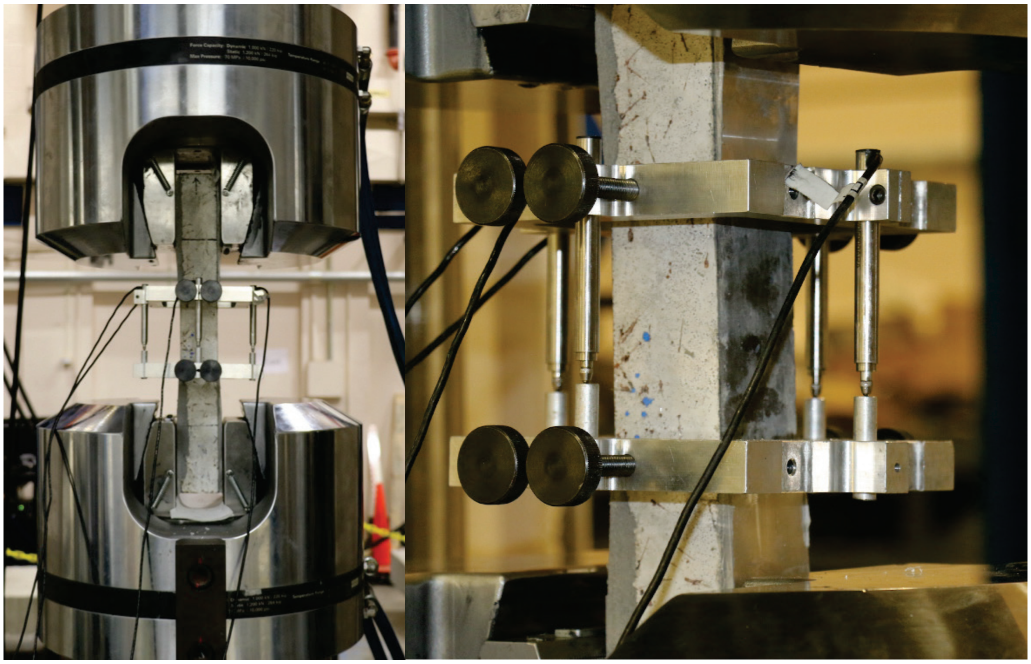


Figure 18. Direct tension testing of ultra-high-performance concrete (UHPC) prism.



4.5 Abrasion Resistance

Abrasion resistance testing was conducted to determine variations in surface wear properties affected by the concrete mixture proportions, steel fiber reinforcement, and material classification. This testing was not intended to provide a quantitative measurement of the length of service that might be expected from a specific mixture; it was used to provide comparative assessments between the mixtures used in this study.

There are no standardized test methods explicitly for determining the abrasion resistance of UHPC or concretes designed to be abrasion resistant. To test abrasion-resistant concretes, some of the standards designed for normal strength concrete permit the application of an increased load, testing duration, or both. Two standardized test methods were selected for use in this study based on the types of abrasion they used and how closely those types represented abrasion action on a navigation lock wall via contact with vessels: ASTM C779, *Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces* (2019c) and ASTM C944, *Standard Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method* (2019b). For each abrasion test method, a 12 in. × 12 in. × 2 in. thick panel was cast, and 4–5 abrasion sites were tested on each panel.

4.5.1 ASTM C779 Core Barrel Method

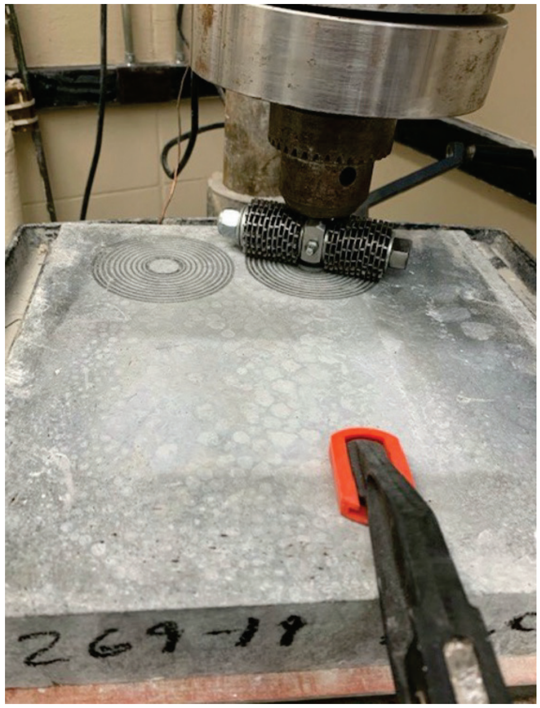
Procedure C of ASTM C779 (2019c) was selected because it involves high-contact stresses, impact, and sliding friction (Figure 19). The method utilizes a modified core barrel resting atop a revolving disk that holds eight 23/32 in. diam steel ball bearings that rotate and abrade the concrete surface with a total load of 27 lbf acting down on the concrete panel. A low velocity water flow washes away spalled or abraded material so that each ball bearing maintains contact with the intact concrete surface. Depth of wear was measured using a calibrated micrometer to the nearest thousandth of an inch and averaged across the wear surface.

Figure 19. ASTM C779 (2019c) core barrel abrasion testing.

4.5.2 ASTM C944 Rotating-Cutter Method

ASTM C944 (2019b) is often used for quality control for highway and bridge surfaces subject to traffic. The rotating cutter is composed of 22 1.5 in. diam dressing wheels symmetrically mounted onto a bolt around a rod end bearing with each dressing wheel separated by a 1 in. washer. This results in an abraded area of 3.25 in. in diameter, creating a specific wear surface, as shown in Figure 20. The rod end bearing is gripped by a drill press that is loaded to exert either a single load of 22 lbf or a double load of 44 lbf. ASTM C944 specifies running the test for three 2 min rounds on the same wear surface and measuring the mass loss to the nearest 0.1 g and the abraded depth after each round.

Figure 20. ASTM C944 (2019b) rotating-cutter abrasion testing.



For this study, the double load was applied to provide a greater differentiation between the behaviors of UHPC and normal strength concrete. The number and duration of abrasion rounds were later increased to five 4 min rounds with the hope of further distinguishing between the performances of the UHPC mixtures Ductal and Lock-Tuf. After each round, the abraded surface was cleaned by using a vacuum, and mass loss was measured. Attempts were made to measure depth of abrasion, but the ruts created by the rotating cutters were irregular, making consistent depth measurement difficult to obtain. Therefore, only mass loss is reported for this test method.

4.6 Impact Resistance

The impact resistance of PAC-5, Lock-Tuf, and Ductal was evaluated using a modified drop-weight impact test described by Badr and Ashour (2005). In this method, a drop hammer style soil compactor meeting ASTMs D1557 (2012a) and D698 (2012b) is modified to produce a line load on a 6 in. diam × 2 in. thick concrete puck between two 1 in. triangular-shaped wedge cutouts on opposite sides of the specimen. The concrete pucks were cut from 6 in. × 12 in. cylinders with the top and bottom 2 in. of the cylinder being discarded. A line load was produced by welding a 2 in. long × ½ in. diam hardened steel rod to the bottom of the 10 lb hammer and dropping the hammer repeatedly from a height of 18 in. until the specimen cracked. The number of blows until first crack (FC) was recorded, and the process was repeated until ultimate rupture (UR). UR is defined as a complete separation of the failure plane in the concrete puck or a separation of at least 10 mm. A failure criterion is provided, stating that only specimens that fail along the line of impact between the two wedge cutouts are accepted and any other failure path is rejected. The intent of this modified method is to provide impact characterization with a reduced coefficient of variation compared to the original ACI drop-weight impact method (ACI Committee 544 1989). Lock-Tuf and PAC-5 samples were cured in the fog room for 28 days, cut to shape, and allowed to air dry for 7 days prior to testing. Ductal samples were steam-cured prior to being placed in the fog room until the time of testing.

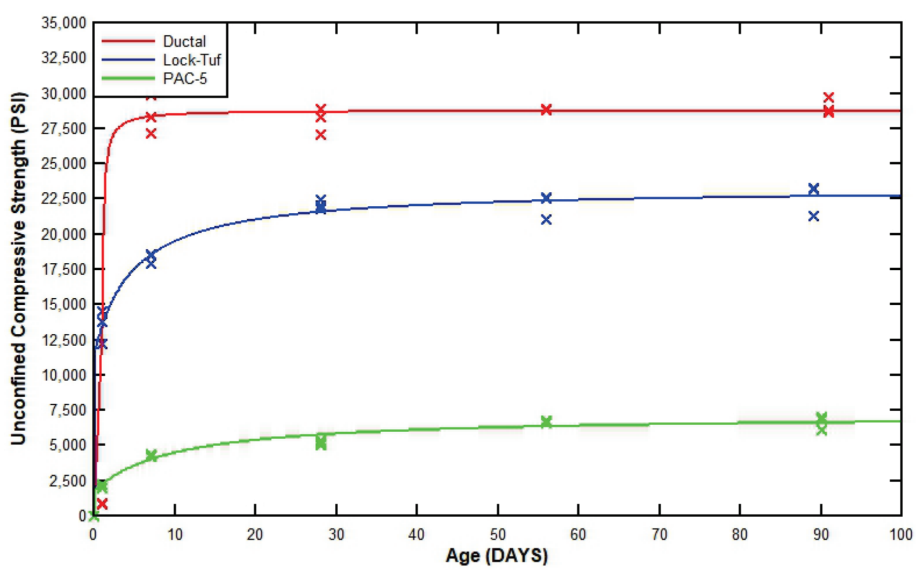
5 Results and Discussion

The results of the experimental test methods are shown in groups. The first group includes what are considered to be the primary concrete mixtures investigated in this study—PAC-5, Lock-Tuf, and Ductal. The second group includes what are considered to be the alternative mixtures—mixtures of Lock-Tuf using alternative fiber types instead of the original OL fibers. In the presentation of results of the alternative mixtures, the result of the original Lock-Tuf mixture is provided as a reference. In these figures, “LT” refers to Lock-Tuf.

5.1 Unconfined Compressive Strength

The compressive strengths from the initial characterization of Ductal, Lock-Tuf, and PAC-5 were measured in accordance with ASTM C39 (2021b) and are presented in Figure 21. Ductal, due to the steam-curing process, rapidly reached an ultimate strength of 28.5 ksi at an age of 7 days and plateaued with no significant strength gain at later ages. Lock-Tuf reached a compressive strength of 18 ksi at 7 days with the wet cure and fly ash inclusion leading to a more rounded strength gain, reaching approximately 22.5 ksi at 90 days. PAC-5 presented a typical normal strength concrete compressive strength gain, reaching an ultimate strength of 6 ksi at 56 days.

Figure 21. Unconfined compressive strengths of Ductal, Lock-Tuf, and PAC-5.

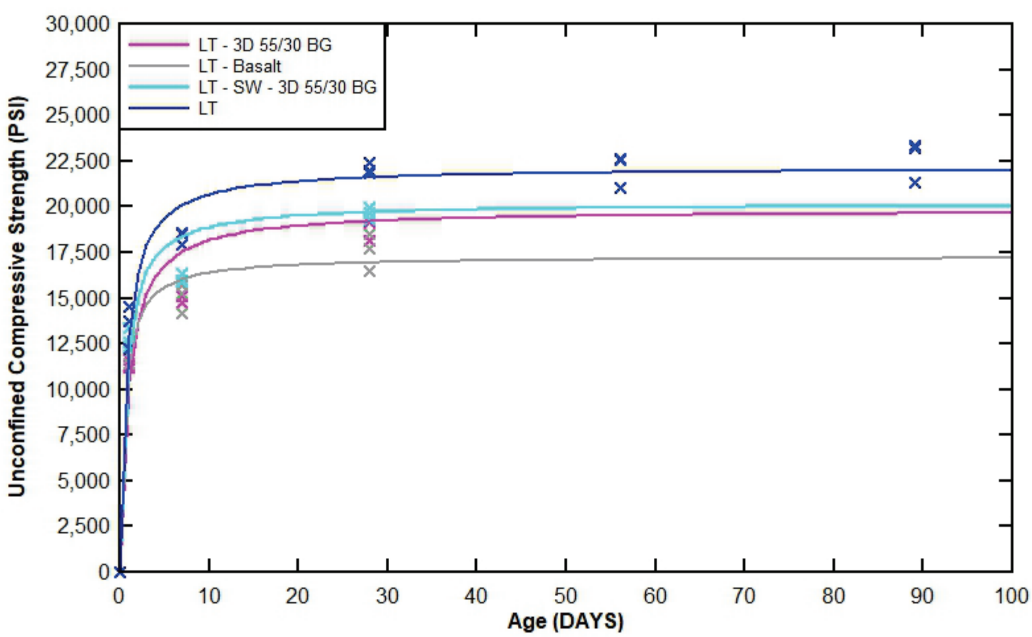


There is a marked difference in 1-day compressive strengths between the two UHPC mixtures: one that is significant for precast manufacturing.

Ductal requires three days of steam curing after casting and exhibits negligible strength after 24 hr, meaning that the panels and forms must remain in place for those three days. Lock-Tuf, however, does not require steam curing and exhibits compressive strengths between 12,500 and 15,000 psi after 24 hr. This rapid strength gain will allow the precast manufacturer to break forms and move the panels out of the casting bed, resulting in more rapid, efficient production than that for Ductal.

The compressive strengths of the three Lock-Tuf mixtures with the alternative fiber types were measured and are presented in Figure 22. The three alternative mixtures maintained the same 2% fiber volume fraction and yielded slightly lower compressive strengths than the original design with the OL fibers. Lock-Tuf with basalt fibers performed the worst with an average compressive strength of approximately 17.5 ksi at 28 days.

Figure 22. Unconfined compressive strengths of Lock-Tuf with alternative fiber types.

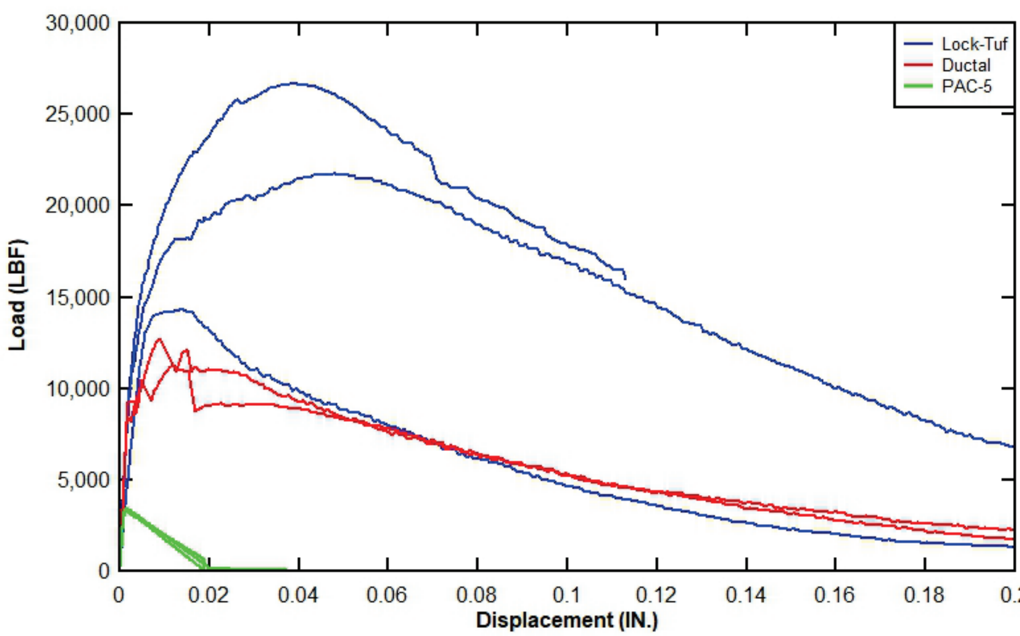


The reductions in compressive strength of the alternative Lock-Tuf mixtures compared to original Lock-Tuf are most likely due to the differences in fiber sizes, aspect ratios, and material. The 3D 55/30 BG fibers are thicker, longer, and have a higher aspect ratio than the OL 13/.20 mm fibers. Other studies have attributed reductions in mechanical properties to these factors (Yoo et al. 2017; Jabbar et al. 2021).

5.2 Flexural Performance

Flexural performance, indicated by the concrete modulus of rupture (MOR), was measured in accordance with ASTM C1609 (2019d), using 6 in. × 6 in. × 21 in. flexural beams tested on an 18 in. span. This testing is presented in a load versus displacement plot shown in Figure 23. The two UHPC mixtures drastically outperformed the normal strength PAC-5 mixture by exhibiting improved flexural strengths with increased postyield load carrying capacity. Lock-Tuf had the highest average peak loads at approximately 18,000 lbf, and Ductal exhibited peak loads at approximately 13,000 lbf. The average peak load exhibited by the PAC-5 mixture was approximately 3,500 lbf. Both UHPC materials demonstrated a deflection-softening behavior that added increased toughness after matrix yielding.

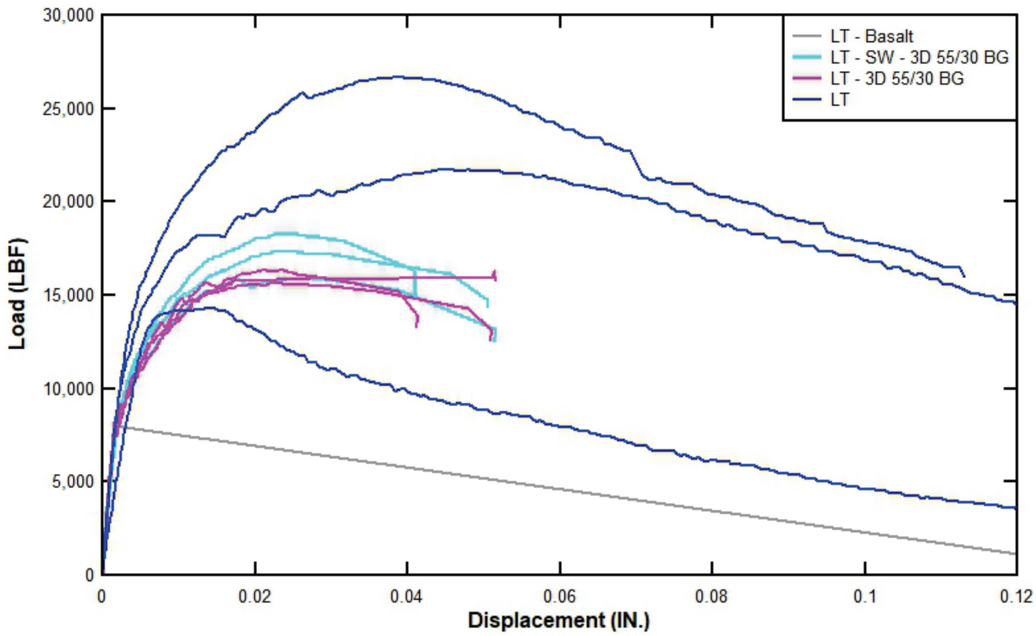
Figure 23. Flexural performance of Lock-Tuf, Ductal, and PAC-5.



The flexural performances of the mixtures with alternative fiber types are presented in Figure 24. All the steel fiber variations achieved similar performance with peak response averaging between 16,000 and 18,000 lbf and deflection-softening behavior. Testing of the 3D 55/30 BG fiber variants, both with and without steel wool fibers, was stopped at a 5% drop in load due to technician error. However, these tests were exhibiting the same postyield behavior as the Lock-Tuf with OL fibers. The basalt fiber variant demonstrated the worst flexural performance in terms of both peak load around 8,000 lbf and postyield load carrying capacity.

There was not an immediate drop in load after yielding, but the energy dissipation capacity decreased much more rapidly than that for the mixtures incorporating steel fibers. The resulting plot appears to show linear behavior after yielding, but it is in fact showing the connection of fewer captured data points after a rapid drop in load carrying capacity.

Figure 24. Flexural performance of Lock-Tuf with alternative fiber types.



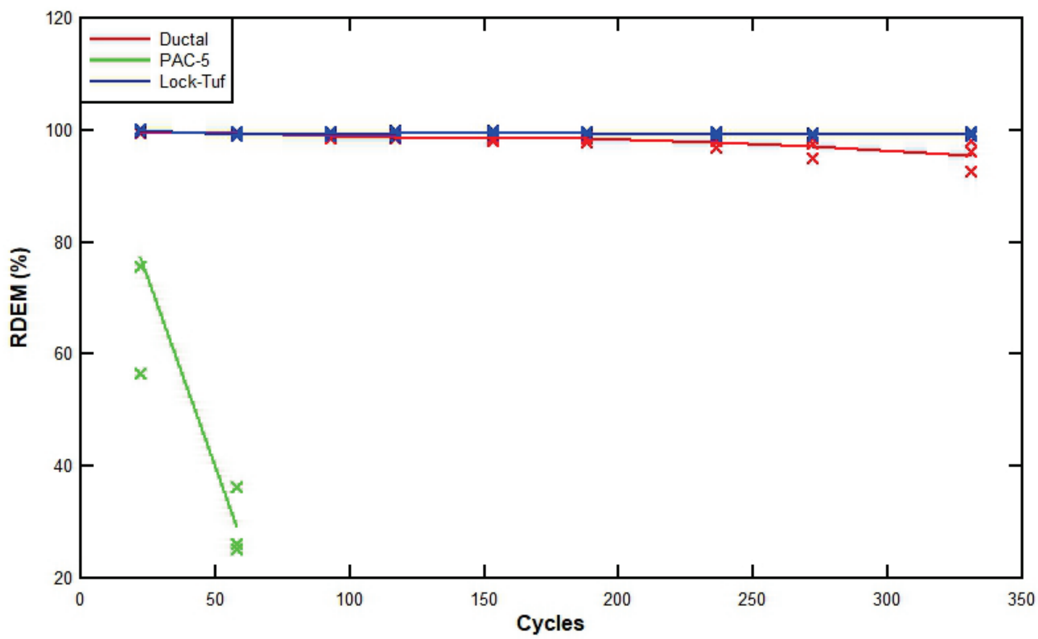
5.3 Freeze–Thaw Cycling

Freeze–thaw testing was conducted for two different purposes in this study. Testing of unreinforced beams was conducted to compare performance of the three primary mixtures—PAC-5, Lock-Tuf, and Ductal. To evaluate the potential of debonding between UHPC and GFRP reinforcement, reinforced Lock-Tuf beams were tested first in freezing and thawing and then in flexural tests. All freeze–thaw testing was conducted in accordance with Procedure A in ASTM C666 (2015).

5.3.1 Unreinforced Beams

The freeze–thaw resistance of PAC-5, Lock-Tuf, and Ductal are presented in Figure 25 in terms of relative dynamic elastic modulus (RDEM) versus number of cycles. PAC-5 had poor performance, failing after approximately 60 cycles. PAC-5 is a non-air-entrained concrete mixture with a typical air content between 2.5% and 3%. It is well known that the addition of air entrainment significantly improves the freeze–thaw performance of normal strength concretes. The intent of this experimentation was to show that, despite the lack of air entrainment, the low permeability of UHPCs leads to exceptional freeze–thaw performance with both UHPCs retaining over 95% of their RDEM and Lock-Tuf showing no measurable difference in RDEM after exposure to 325 freeze–thaw cycles.

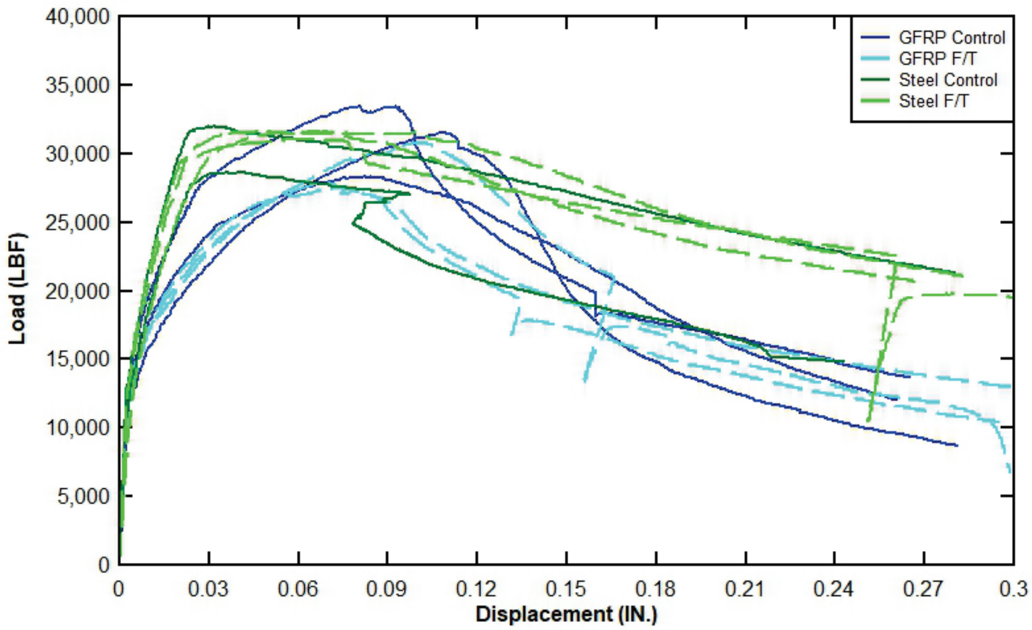
Figure 25. ASTM C666 freeze–thaw performance of PAC-5, Lock-Tuf, and Ductal.



5.3.2 Reinforced Beams

After 300 cycles, each Lock-Tuf beam containing either steel or GFRP reinforcement had minimal loss in RDEM. These beams were then subjected to ASTM C1609 (2019d) flexural performance testing with the single reinforcement bar being placed along the tensile face. These tests are presented in Figure 26, and results show no substantial difference in peak loads or displacement prior to failure.

Figure 26. Flexural performance of reinforced Lock-Tuf beams after freeze–thaw cycling.



The main difference in the load displacement curves is the stiffness between initial cracking and peak load of the steel versus GFRP reinforcement. The steel reinforcement is stiffer, with a higher elastic modulus, resulting in a steeper, nearly linear curve up to the point of yield. The GFRP bar is not as stiff, resulting in a more rounded curve after cracking and up to peak load, which happened at a higher displacement than that of the steel reinforced beams.

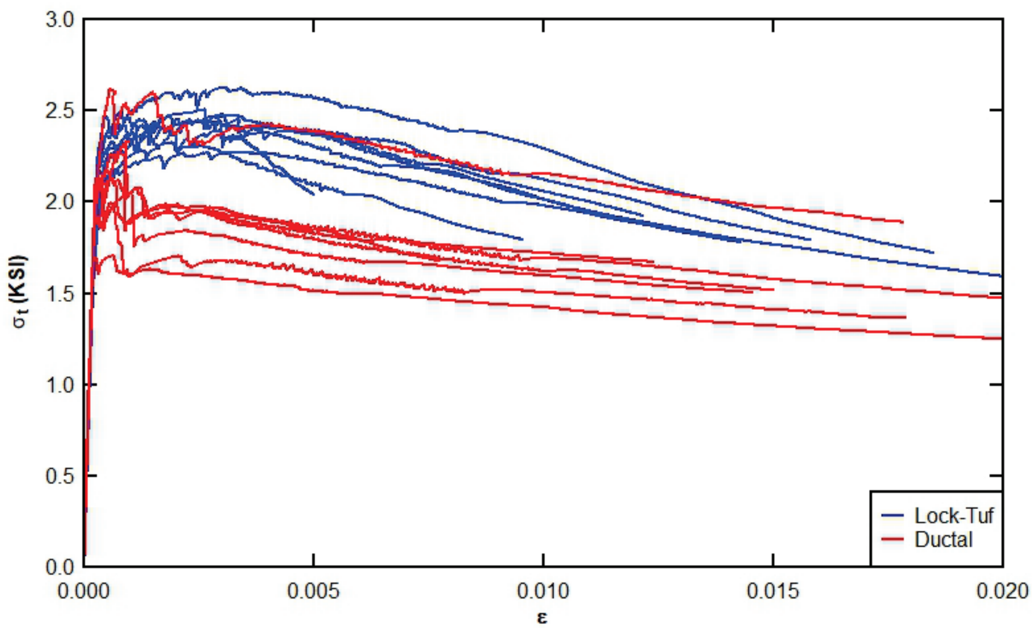
Interestingly, there was essentially no difference between the unconditioned control specimens and those that were subjected to freeze–thaw cycles for both types of reinforcement. Despite severe freeze–thaw exposure, the reinforcement in the Lock-Tuf concrete showed no signs of debonding. According to this study, GFRP is an acceptable alternative to steel as reinforcement in Lock-Tuf.

5.4 Direct Tension

Direct tension tests were conducted in accordance with the FHWA method (Graybeal and Baby 2013), and results are shown in Figure 27. Ductal samples first cracked at 2 ksi, followed by a drop in load that led into strain-softening behavior up to an average of 1.6% strain. Lock-Tuf first cracked at approximately 2.2 ksi with a brief strain-hardening phase up to

0.3% strain at a peak stress of 2.4 ksi. The slight strain-hardening phase then transitioned to a strain-softening phase up to an average of 1.45% strain. In this test method, Lock-Tuf generally outperformed Ductal.

Figure 27. Direct tension behavior of Lock-Tuf and Ductal.



5.5 Abrasion Resistance

Abrasion resistance testing was conducted according to two different test methods, ASTM C779 (2019c) and ASTM C944 (2019b). Both methods utilized a 12 in. × 12 in. × 2 in. thick concrete panel with three to five different abrasion sites per panel.

5.5.1 ASTM C779 Core Barrel Method

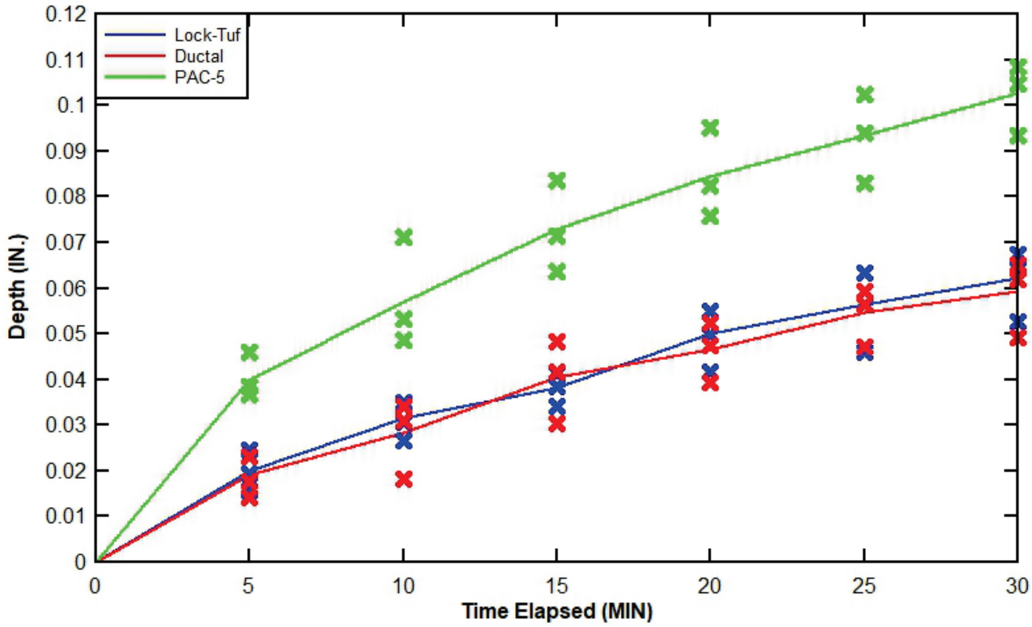
Figure 28 shows the typical abrasion damage incurred by PAC-5 and Lock-Tuf under ASTM C779 (2019c) abrading conditions. Each contact area was subjected to 5 min intervals of abrasion with the modified core-barrel abrader. The figure shows the abraded surface after all six 5 min rounds were completed on each test site.

Figure 28. ASTM C779 (2019c) abraded surfaces of PAC-5 (*left*) and Lock-Tuf (*right*).

The results of this testing for PAC-5, Lock-Tuf, and Ductal are presented in Figure 29 and show abraded depth versus time elapsed. PAC-5 exhibited the worst abrasion resistance with an abraded depth 1.7 × deeper than either of the UHPC materials. Lock-Tuf and Ductal performed equally well in this method with a maximum abraded depth of approximately 0.06 in.

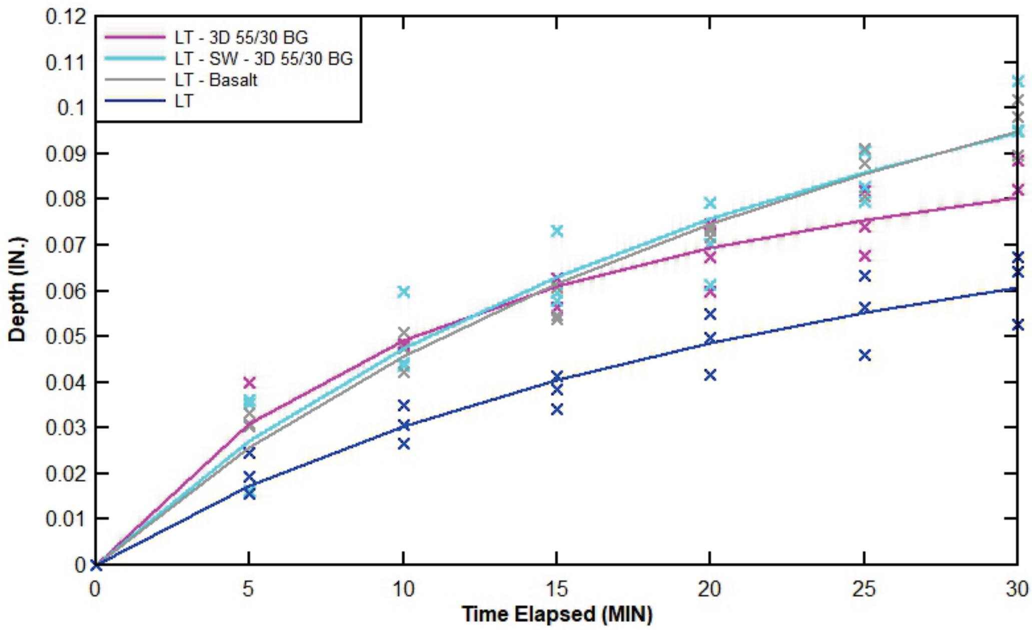
All three abrasion depth curves have a parabolic shape about the *x*-axis. The slope of the curve for the first 5 min interval is steeper than that for the next 5 min interval. Each subsequent interval generally has a shallower abrasion depth than the interval prior. In the case of PAC-5, this is due to the abrader's grinding past the paste-rich outer surface and eventually making contact with more and more fine and coarse aggregates. For the two UHPC mixtures, the abrader grinded past the paste-rich outer surface and came in contact with fine aggregates and steel fibers. The improved abrasion resistance exhibited by the UHPCs is due to the improved matrix densities of the concretes and the high-volume fractions of steel fibers.

Figure 29. ASTM C779 (2019c) abrasion resistance of PAC-5, Lock-Tuf, and Ductal.



The ASTM C779 (2019c) abrasion resistance for the Lock-Tuf with alternative fiber types is presented in Figure 30. All three alternative fiber types demonstrated a higher abrasion depth after each interval than Lock-Tuf with the OL fibers. The basalt fibers did not exhibit the same hardness as the steel fibers and were, therefore, less capable of resisting the abrasion. The mixtures using the larger 3D 55/30 BG fibers had lower fiber densities than the mixtures using the OL fibers when proportioned at a constant volume fraction of 2%. This resulted in fewer fibers at the contact surfaces in the alternative Lock-Tuf mixtures, resulting in poorer abrasion resistance compared to the original Lock-Tuf mixture.

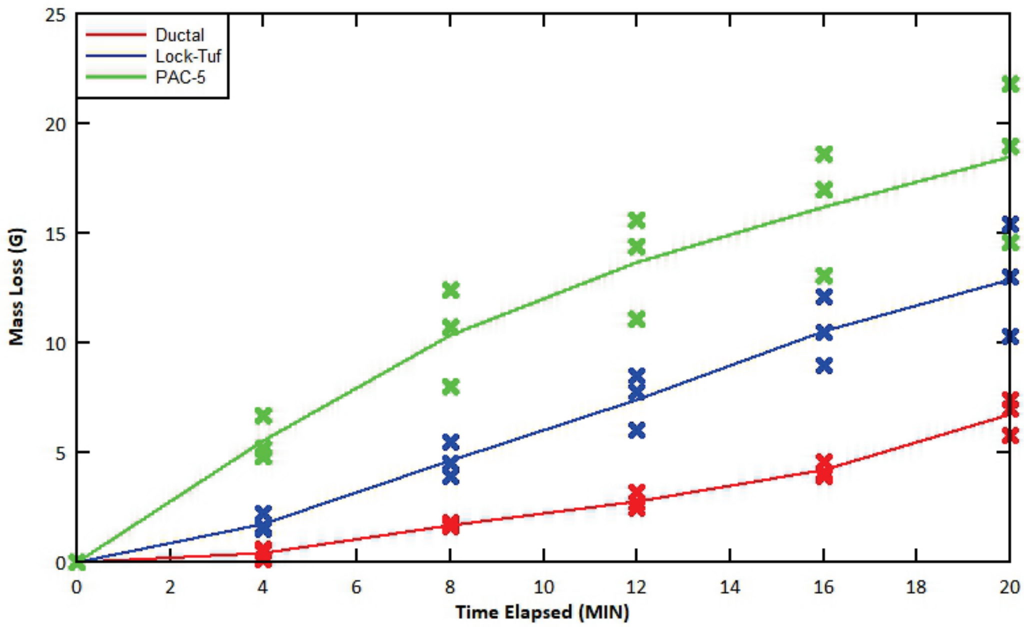
Figure 30. ASTM C779 (2019c) abrasion resistance of Lock-Tuf with alternative fiber types.



5.5.2 ASTM C944 Rotating-Cutter Method

Results of abrasion resistance measured by ASTM C944 (2019b) via the rotating-cutter method are presented in Figure 31. The cutter was composed of 22 cutter wheels rotating about a vertical axis with 44 lb of force acting down on the panel. Five 4 min rounds were recorded at three different abrasion sites on a single concrete panel with the abraded surfaces cleaned by vacuum between each round. Mass loss was recorded after each round. PAC-5 had the highest mass loss after all five rounds had been completed, with a total mass loss of 18.5 g. Ductal had the highest abrasion resistance in this method, losing an average of only 6 g of mass—approximately 1/3 of the mass loss exhibited by PAC-5. Lock-Tuf had a mass loss of approximately 2/3 of that of PAC-5, with a total loss of about 12.5 g. Ductal outperformed Lock-Tuf in this test method. Graybeal (2006) documented similar behavior between UHPCs in ASTM C944 (2019b). Those that had been steam-cured exhibited higher abrasion resistance than those UHPCs that had not. This is believed to the result of a more durable mechanical matrix achieved during steam curing.

Figure 31. ASTM C944 (2019b) abrasion resistance of PAC-5, Lock-Tuf, and Ductal.

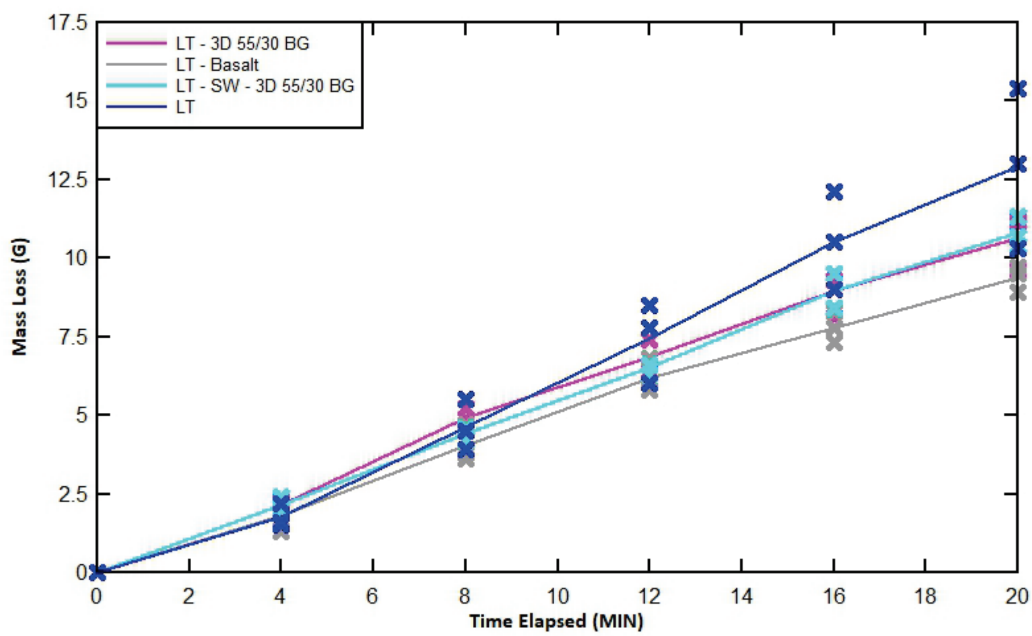


The typical damage created by this method is shown on a Lock-Tuf panel in Figure 32. The damage was characterized by concentric rings of wear around the central axis. The washers between the dressing wheels, as prescribed in the method, were large enough to create additional ruts between each of the rings, thus making a depth of wear measurement hard to obtain. In future work, using a washer with a smaller thickness could potentially prevent this.

Figure 32. Typical abrasion of Lock-Tuf in ASTM C944 (2019b) rotating-cutter method.

The results of rotating-cutter abrasion resistance for Lock-Tuf with alternative fiber types is presented in Figure 33. The performance of mixtures using alternative fiber types is marginally better than that of the original Lock-Tuf, with total mass loss of around 10 g each. These results indicate behavior opposite to the alternate fiber results from the C779 (2019c) test method, which showed better performance of the original Lock-Tuf mixture. The dressing wheels in this method have a smaller surface contact area than the ball bearings utilized in ASTM C779. Also, the concentric rings created do not abrade past the paste-rich outer surface as well as the ball bearings do. Therefore, the steel fibers have less of an effect on abrasion resistance in this method than in the core barrel method.

Figure 33. ASTM C944 (2019b) abrasion resistance of Lock-Tuf with alternative fiber types.



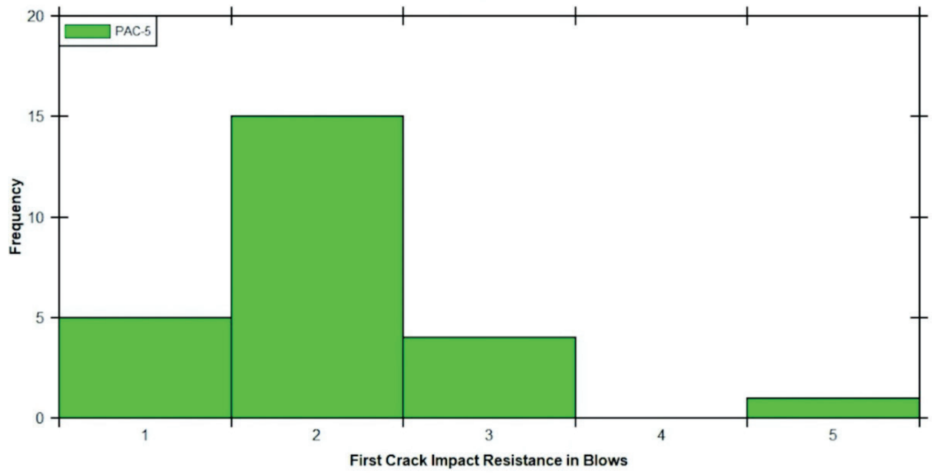
5.6 Impact Resistance

The impact resistance of 6 in. diam × 2 in. thick notched concrete pucks were measured according to the modified ACI drop-weight impact test described by Badr and Ashour (2005). The test method involves counting the number of impacts, or blows, until FC and UR of the samples. A Lock-Tuf specimen tested to UR is shown in Figure 34. Results are presented here in histograms, and the complete data set is presented in Appendix D: Impact Resistance Data. In all, 28 samples of Pac-5 and 30 samples each of Lock-Tuf and Ductal were tested.

Figure 34. Impact test of a Lock-Tuf sample.

The number of blows until FC for PAC-5 is shown in Figure 35. Almost all PAC-5 specimens showed cracking within the first three blows, with only one sample not cracking until the fifth blow.

Figure 35. Number of blows until first crack (FC) of PAC-5 samples in impact tests.



The number of blows until FC for Lock-Tuf and Ductal are presented in Figure 36. In total, two Lock-Tuf specimens and four Ductal specimens did not reach FC within 200 blows. They instead exhibited cratering with no visible cracks in the area between the notched sections of the puck.

Figure 36. Number of blows until FC of Lock-Tuf and Ductal samples in impact tests.

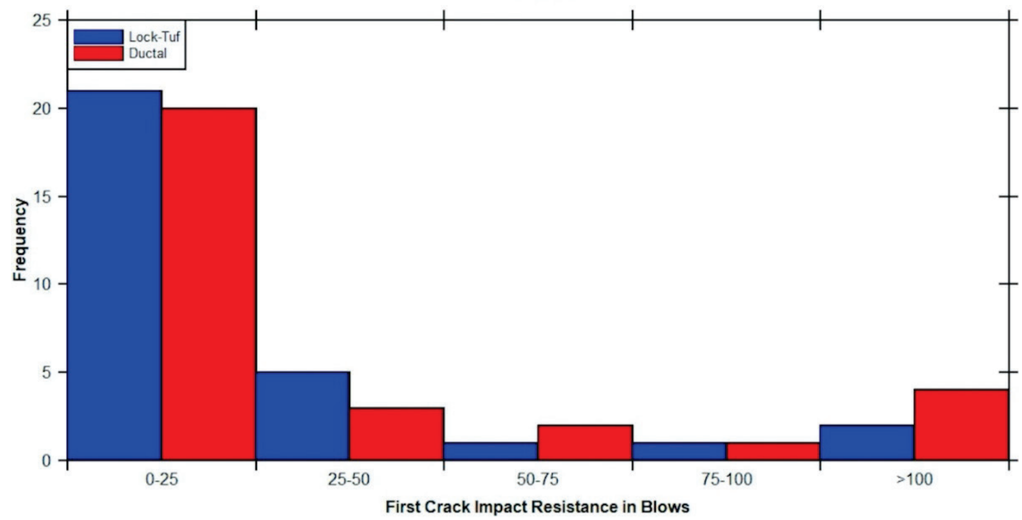
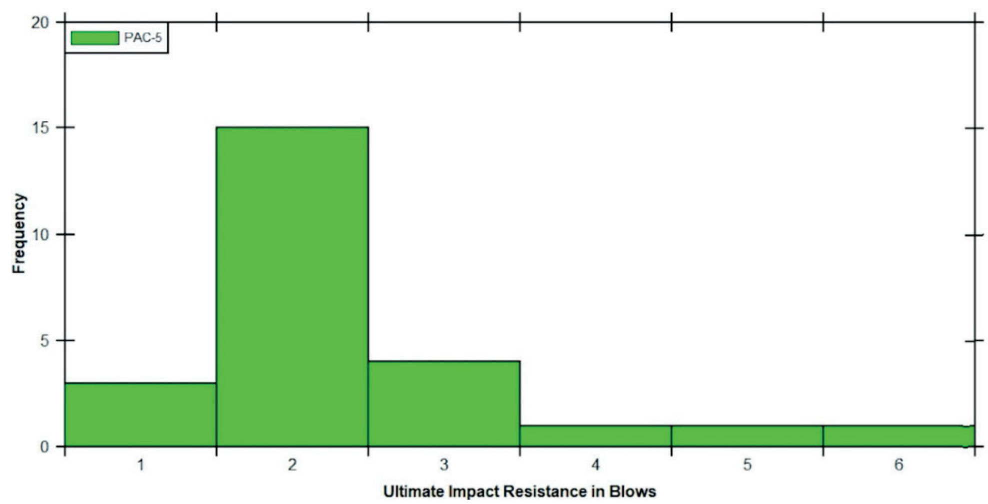


Figure 37 shows the number of blows until UR for PAC-5 specimens in impact tests. The majority of PAC-5 specimens reached UR after two blows. These air-dried specimens had low compressive strength and small rounded aggregates, properties that provided little impact resistance to the loads used in this test method. The majority of these specimens reached FC and UR criteria on the same blow, transitioning from no cracking to UR instantly. This is indicative of uncontrolled fracture growth and brittle material response.

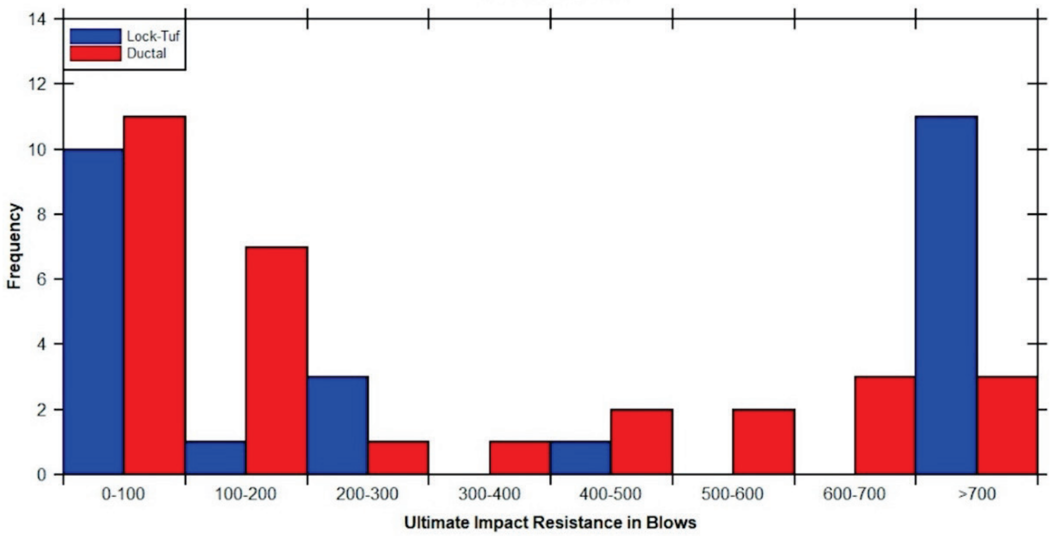
Figure 37. Number of blows until ultimate rupture (UR) of PAC-5 samples in impact tests.



The UHPC specimens resisted a higher range of blows to UR, as shown in Figure 38. The scatter is likely due to random fiber orientation, with some specimens having more fibers bridging the fracture plane than others, and the presence of flaws through the fracture plane in some specimens.

Eleven Lock-Tuf specimens and three Ductal specimens failed to reach UR in 725 blows. Testing for these specimens was stopped, and the final values were recorded as UR despite the specimens not meeting the failure criterion.

Figure 38. Number of blows until UR of Lock-Tuf and Ductal samples in impact tests.



The modified ACI impact test used in this study was created to provide improved statistical reliability compared to the original ACI impact test method (ACI Committee 544 1989). Badr and Ashour (2005) reported a range of coefficients of variation of 48.7 to 61.4% using the original test method and 34.3 to 40.7% using their proposed modified method. The coefficients of variation obtained in this study were higher than those ranges, particularly for the UHPCs. Table 2 provides a summary of the impact testing results along with statistical data.

Table 2. Summary of impact testing results with statistical data.

Parameter	PAC-5		Lock-Tuf		Ductal	
Damage Type	FC	UR	FC	UR	FC	UR
Average Blows	2	2	29	377	45	268
Standard Deviation	0.9	1.2	49.3	314.0	120.8	253.8
Coefficient of Variation	41.5%	48.1%	169.8%	83.2%	150.2%	94.7%
Average Difference in Blows from FC to UR	–	0	–	348	–	223

FC = first crack; UR = ultimate rupture

The coefficients of variation were lowest for the PAC-5 concrete in testing to both FC and UR. The FC coefficients of variation for both Lock-Tuf and Ductal were greater than 150%, and the UR coefficients of variation were greater than 83%, indicating that this method does not yield very reproducible results. Thus, conclusions beyond the obvious differences in performance between normal strength concrete and the UHPCs are difficult to establish. The high variance in these tests, particularly for the UHPCs, is likely due to random fiber orientation and volume in the failure plane between the two triangular notches.

6 Conclusions

This study evaluated the performance of an ERDC-developed, nonproprietary UHPC (Lock-Tuf) in small-scale tests and compared it to that of a normal strength concrete (PAC-5) and a proprietary UHPC (Ductal) to determine its potential for use as a lock wall repair material. Test methods included unconfined compressive strength, flexural performance, resistance to freeze–thaw cycling, reinforcement bonding, direct tension, abrasion resistance, and impact resistance. The conclusions of this research can be summarized as follows:

- It is possible to design a nonproprietary UHPC using materials sourced from a nearby precast concrete plant, some of which may not traditionally be included in UHPCs (i.e., Type III cement). In doing so, the higher cost and logistical burden associated with using proprietary UHPCs can be avoided.
- Both Lock-Tuf and Ductal outperformed PAC-5 in all test methods used in this study, suggesting that UHPC is a better material than normal strength concrete for the proposed application in which durability and impact and abrasion resistance are critical.
- Overall, Lock-Tuf performed as well as, or better than, Ductal in the test methods used in this study. Because, unlike Ductal, Lock-Tuf does not require steam curing, precast manufacturers will be able to quickly and efficiently produce panels using nonproprietary UHPC while meeting strength and durability requirements for the intended application.
- Tests for reinforcement bonding showed significant differences in neither ASTM C666 (2015) control versus conditioned specimens nor in conventional steel versus GFRP-reinforced specimens after flexural testing. This has two implications: (1) UHPC is highly resistant to the effects of freeze–thaw cycling, and (2) GFRP is an acceptable material for internal reinforcement of UHPC panels.
- The two abrasion test methods used in this study produced results with conflicting outcomes. In the ASTM C779 core barrel method (2019c), Lock-Tuf and Ductal behaved nearly the same, and Lock-Tuf with OL fibers outperformed Lock-Tuf with alternative fiber types. However, results of the ASTM C944 rotating-cutter method (2019b) showed a discernable difference between Lock-Tuf and Ductal and better performance from the Lock-Tuf mixtures with alternative fibers compared to original Lock-Tuf. Steam curing has been shown to affect ASTM C944 (2019b) results when comparing UHPC mixtures that use the

same materials and proportions. It is unknown whether curing method has an effect on ASTM C779 results (2019c), but it did not appear so in this study. The differences in performance between the Lock-Tuf mixtures may be attributed to the differences in abrasion action between the two test methods. It is possible that the rotating-cutter was able to pull out the comparatively small OL fibers as it rotated.

- There is a need for reproducible, small-scale test methods for impact resistance of UHPC and other concretes designed to be impact resistant. The large coefficients of variation produced by the modified ACI drop-weight method used in this study prohibited the drawing of meaningful conclusions.

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Appendix A: Lock-Tuf Trial Mixture Proportions

Table A-1. Lock-Tuf trial mixture proportions.

Lock-Tuf Trial Mixtures (PCY)				
Trial	Lock-Tuf T1	Lock-Tuf T2	Lock-Tuf T3	Lock-Tuf T4
w/c+m	0.174	0.172	0.18	0.18
Cement (lb)	1,243.9	1,077.5	1,479.3	1,294.5
Silica Fume (lb)	210.7	349.8	289.2	253.0
Fly Ash (lb)	134.1	307.2	494.9	433.0
Sand (lb)	2,017.6	1,757.0	1,143.1	1,532.7
Steel Fiber (lb)	264.6	264.6	264.6	264.6
Water (lb)	256.4	277.6	407.4	356.5
HRWRA (lb)	31.2	32.1	34.4	30.1

Table A-2. Lock-Tuf trial mixture details.

Lock-Tuf Trial Mixtures Details				
Trial	Lock-Tuf T1	Lock-Tuf T2	Lock-Tuf T3	Lock-Tuf T4
A/B Ratio	1.27	1.01	0.51	0.77
% Silica Fume of Cementitious Volume	17.25	25	16.4	16.4
% Fly Ash of Cementitious Volume	10	20	25	25
% Steel Fibers Volume	2	2	2	2
HRWRA (Fl. Oz/100 wt.)	27.6	26.1	14.3	16.08
Total Weight of Cementitious	1,588.7	1,734.6	2,263.3	1,980.50

Appendix B: Lock-Tuf Mill Certificates

Holcim

NSF Certified to NSF/ANSI 61 & 372

Material Certification Report

Material: Portland Cement
Type: III

Test Period: 01-Feb-2021 to 28-Feb-2021
Date Issued: 12-Mar-2021

Certification

This cement meets the specifications of ASTM C150 and AASHTO M85 for Type III cement.

General Information

Supplier:	Holcim (US) Inc. d/b/a LafargeHolcim US	Source Location:	Theodore Plant
Address:	8700 West Bryn Mawr Ave Chicago, IL 60631		3051 Hamilton Blvd Theodore, AL 36582
Contact:		Contact:	Alissa Collins / (251) 443-1290

The following is based on average test data during the test period. The data is typical of product shipped from this source; individual shipments may vary.

Test Data on ASTM Standard Requirements

Chemical			Physical		
Item	**Limit [1]**	**Result**	**Item**	**Limit [1]**	**Result**
SiO_2 (%)	-	20.7	Air Content (%)	12 max	6
Al_2O_3 (%)	-	4.7	Blaine Fineness (m^2/kg)	-	576
Fe_2O_3 (%)	-	3.4			
CaO (%)	-	65.2	Autoclave Expansion (%) (C151)	0.80 max	0.00
MgO (%)	6.0 max	1.1	Compressive Strength MPa (psi)		
SO_3 (%) [2]	3.5 max	4.0	1 day	12.0 (1740) min	24.0 (3480)
Loss on Ignition (%) [5]	3.5 max	1.5	3 day	24.0 (3480) min	34.4 (4990)
Insoluble Residue (%)	1.50 max	0.09	28 day (previous month's data)	-	51.5 (7470)
CO_2 (%)	-	0.4			
$CaCO_3$ in Limestone (%)	70 min	95	Initial Vicat (minutes)	45-375	75
Potential Phase Compositions [3]:					
C_3S (%)	-	60	Mortar Bar Expansion (%) (C1038)	0.020 max	0.009
C_2S (%)	-	14			
C_3A (%)	15 max	7			
C_4AF (%)	-	10			

Test Data on ASTM Optional Requirements

Chemical			Physical		
Item	**Limit [1]**	**Result**	**Item**	**Limit [1]**	**Result**
Equivalent Alkalies (%)	-	0.20			

Notes (*1-9)

1 - Dashes in the Limit / Result columns mean Not Applicable.
2 - It is permissible to exceed the specification limit provided that ASTM C1038 Mortar Bar Expansion does not exceed 0.020% at 14 days.
3 - Adjusted per Annex A1.6 of ASTM C150 and AASHTO M85.
5 - Limit = 3.0 when limestone is not an ingredient in the final cement product

Additional Data

Item	Limestone	Inorganic Processing Addition	Base Cement Phase Composition	Result
Amount (%)	0.8	-	C_3S (%)	60
SiO_2 (%)	5.6	-	C_2S (%)	14
Al_2O_3 (%)	0.2	-	C_3A (%)	7
Fe_2O_3 (%)	0.2	-	C_4AF (%)	10
CaO (%)	51.1	-		
SO_3 (%)	0.1	-		

Printed: 3/12/2021 1:21:54 PM

Version: 180412

Alissa Collins

Alissa Collins,
Quality Manager

Technical Silica Co.

D-1000 Silica Fume

Product Data

Bright white amorphous silica fume with low impurity level.

Used in a variety of cementitious and refractory applications to increase strength and durability. See technicalco.com for more details.

Packaging: 500 kg bulk bags, 50 lb bags

	Typical %
SiO_2	88-90
$ZrO_2 + HfO_2$	4-6
Al_2O_3	4-6
TiO_2	0.04
C	0.20
Fe_2O_3	0.10
CaO	0.04
Na_2O	0.06
MgO	0.07
Moisture	0.30
+325 mesh	0.24
pH	2.5-4.5
Bulk Density (lbs/ft^3)	21-31
Color	White

Technical Silica Company
2250 North Druid Hills Rd.
Suite 235
Atlanta, GA 30329
Phone: 404-321-0460
Fax: 404-633-0799
www.technicalco.com

Jonathan Gavant
Sales Manager
Email: jonathan@technicalco.com
Cell: 404-844-7324

Made in the USA
Premium Quality

BORAL RESOURCES

Materials Testing & Research Facility
2650 Old State Hwy 113
Taylorsville, GA 30178
770-684-0102

ASTM C618 / AASHTO M295 Testing of
Barry Class F Fly Ash

Sample Date:	12/1 - 12/31/20	**Report Date:**	2/9/2021
Sample Type:	Monthly	**MTRF ID:**	40BR
Sample ID:	Unit 5 Silo		

Chemical Analysis	**Results**		**ASTM Limit Class F / C**	**AASHTO Limit Class F / C**
Silicon Dioxide (SiO2)	62.31	%		
Aluminum Oxide (Al2O3)	19.42	%		
Iron Oxide (Fe2O3)	7.19	%		
Sum (SiO2+Al2O3+Fe2O3)	88.92	%	50.0 min	50.0 min
Sulfur Trioxide (SO3)	0.50	%	5.0 max	5.0 max
Calcium Oxide (CaO)	2.84	%	18.0 max / >18.0	18.0 max / >18.0
Magnesium Oxide (MgO)	1.60	%		
Sodium Oxide (Na2O)	2.04	%		
Potassium Oxide (K2O)	2.04	%		
Sodium Oxide Equivalent (Na2O+0.658K2O)	3.38	%		
Moisture	0.09	%	3.0 max	3.0 max
Loss on Ignition	0.46	%	6.0 max	5.0 max

Physical Analysis				
Fineness, % retained on 45-µm sieve	12.26	%	34 max	34 max
Strength Activity Index - 7 or 28 day requirement				
7 day, % of control	84	%	75 min	75 min
28 day, % of control	90	%	75 min	75 min
Water Requirement, % control	93	%	105 max	105 max
Autoclave Soundness	0.02	%	0.8 max	0.8 max
Density	2.43	g/cm3		

The test data listed herein was generated by applicable ASTM methods. The reported results pertain only to the sample(s) or lot(s) tested. This report cannot be reproduced without permission from Boral Resources. This material meets the requirements of Florida DOT 929 specification and Mississippi DOT Section 714.05.2.

Christy Sieg
Christy Sieg
Lab Manager

AASHTO ACCREDITED

American Engineering Testing, Inc.

St. Paul	Albertville
550 Cleveland Ave N	5548 Barthel Ind Dr, Ste 500
St. Paul, MN 55114	Albertville, MN 55301
(651) 659-9001	(763) 428-5573
Toll Free: (800) 972-6364	www.amengtest.com

Material Test Report

Report No: MAT:20-19715-S1
Issue No: 1

Client: BURNS COOLEY DENNIS, INC.   **CC:**

Project: BCD Project No. 200444

Job No: 24-21478

Date of Issue: 8/27/2020
Reviewed By: Jesse Sich
MNDOT Tech ID#17564

Sample Details

Sample ID	20-19715-S1
Date Sampled	8/21/2020
Source	P & R White Sand
Material	Concrete Sand
Specification	C33 FA Spec
Sampling Method	Supplied By Client
General Location	BCD Project Number 200444
Location	Sand
Date Submitted	8/21/2020

Other Test Results

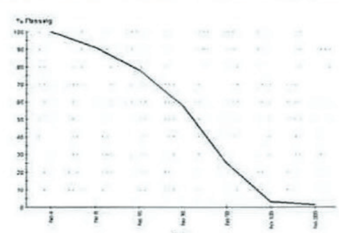
Description	Method	Result	Limits
Mass of Test Sample (g)	ASTM C 123	200.0	
Nominal Maximum Size (in)		Fine Aggregate	
Type of Heavy Liquid		LMT	
Heavy Liquid Specific Gravity		2.00	
Lightweight Particles (%)		0.0	
Shale Particles (%)		0.0	
Coal and Lignite Particles (%)	✔	0.0	≤1.0
Chert Particles (%)		0.0	
Mass of Test Sample (g)	ASTM C 123	200.0	
Nominal Maximum Size (in)		Fine Aggregate	
Type of Heavy Liquid		LMT	
Heavy Liquid Specific Gravity		2.40	
Lightweight Particles (%)		0.7	
Shale Particles (%)		0.0	
Coal and Lignite Particles (%)	✔	0.0	≤1.0
Chert Particles (%)		0.7	
Fineness Modulus	ASTM C 136, ASTM C 117	2.45	

Particle Size Distribution

Method: ASTM C 136, ASTM C 117
Drying by: Oven
Date Tested: 8/24/2020
Tested By: 20-St. Paul

Sieve Size	% Passing	Limits
No.4 (4.75mm)	100	
No.8 (2.36mm)	91	
No.16 (1.18mm)	78	
No.30 (600µm)	58	
No.50 (300µm)	25	
No.100 (150µm)	3	
No.200 (75µm)	1.2	

Chart



Comments

N/A

BURNS COOLEY DENNIS, INC.
CONSTRUCTION MATERIALS AND ENGINEERING TESTING SERVICES

278 COMMERCE PARK DRIVE
RIDGELAND, MISSISSIPPI 39157

Phone: (601) 856-2332
Fax: (601) 856-3552

To: Tindall Corporation
11450 Saracennia Road
Moss Point, Mississippi 395643

Report Date: 1/24/2020

BCD Project No.: 200037

Attn: Tom Moore

Project: Aggregate Characteristics Testing 2020 PO. MS2009

SOURCE AND SAMPLING INFORMATION

Aggregate Source:	P & R Natural Sand	Aggregate Size:	Fine
Sampling Location:			Fine
Sampled By:	Client		
Date Received:	1/17/2020	Tested By:	J. Seay
BCD Lab No:	1	Date Tested:	1/23/2020

SPECIFIC GRAVITY AND ABSORPTION OF FINE AGGREGATE (ASTM C128)

A = mass of oven-dry test sample in air (0.1 g)	**500.8**
B = mass of pycnometer filled with water (0.1 g)	**645.1**
S = mass of saturated-surface-dry specimen (0.1 g)	**503.0**
C = mass of pycnometer with SSD specimen and water to calibration mark (0.1 g)	**954.3**
Bulk Specific Gravity (Dry) **Bulk sp gr = A / (B+S-C)**	**2.584**
Bulk Specific Gravity (Saturated-Surface-Dry) **Bulk sp gr (saturated-surface-dry) = S / (B+S-C)**	**2.595**
Absorption, percent **= [(S-A) / A] X 100**	**0.44**

REPORTED BY: CMT Manager Engineer

3		
	03 30 00	Cast-in-Place Concrete
	03 40 00	Precast Concrete
	03 70 00	Mass Concrete

MASTER®
»BUILDERS
SOLUTIONS

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 (130-780 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.**

A brand of MBCC GROUP

3	03 30 00	Cast-in-Place Concrete
	03 40 00	Precast Concrete
	03 70 00	Mass Concrete
4	04 05 16	Masonry Grouting

MASTER®
»BUILDERS
SOLUTIONS

MasterSure® Z 60

Workability-Retaining Admixture

Description

MasterSure Z 60 admixture is a workability-retaining admixture that provides flexible degrees of slump retention without retardation. MasterSure Z 60 admixture provides concrete producers a cost-effective means of maintaining consistency between loads of concrete with respect to slump, workability and air content. MasterSure Z 60 admixture meets ASTM C 494/C 494M requirements for Type S, Specific Performance, admixtures.

Applications

Recommended for use in:

- Concrete with varied slump retention requirements
- Concrete mixtures utilizing supplementary cementitious materials
- Concrete where high flowability, increased stability and durability are needed
- Production of self-consolidating concrete (SCC) mixtures

Features

- Provides flexible degrees of slump and workability retention without retardation
- Can be used in low- to high-slump concrete mixtures, including self-consolidating concrete
- Can be used alone or in combination with MasterPozzolith®, MasterPolyheed® or MasterGlenium® normal, mid-range and high-range water-reducing admixtures
- Improved early- and late-age compressive strengths

Benefits

- Minimizes the need for jobsite slump adjustment using water or high-range water-reducing admixture
- Provides consistency in air-entrainment, slump, workability and strength
- Fewer rejected loads and better customer satisfaction due to consistent quality of concrete
- Faster truck turn-around time
- Expanded concrete delivery range
- Provides concrete producers with ability to consistently produce and deliver quality concrete mixtures

BEKAERT
better together

Dramix® OL

OL 13/.20

Length Diameter

DATASHEET

Characteristics

Material properties

Bright, High Carbon wire

Nom. tensile strength:	2.750 (N/mm²)
Young's modulus:	200.000 (N/mm²)

Geometry

Length (l): 13 mm

Diameter (d): 0,2 mm

Fibre network

220.394 m/m³ at 60 kg/m³
282.556 fibres/kg

Safety

Product certificates*

CE

Product conformity

Dramix® conforms to ASTM A820, EN 14889-1.

System certificates

TÜV NORD ISO 9001 TÜV NORD ISO 14001

All Dramix® plants are ISO 9001 and ISO 14001 certified.

Packaging

kg / bag: 15
bags / pallet: 50
kg / pallet: 750

Handling

KEEP DRY

NO STACKING

DRAMIX® OL 13/.20

THE HIGH PERFORMANT STRENGTH

Dramix® OL is a high performant fiber to create optimal ductility in UHPC.

BEKAERT CONSTRUCTION SUPPORT

You can count on our support for each step of your project, from concept design to on-site quality support. Our services include recommendations on slab design, construction detailing, concrete optimization and automatic total quality control procedures.
We are also happy to share our knowledge with you and your team. Feel free to ask us for a workshop or training on the topic of steel fiber reinforcement in your offices.

For recommendations on handling, dosing and mixing visit www.bekaert.com/dosingdramix.
Any other specific document or certificate can be found on www.bekaert.com/doc/Dramix_OL.

Appendix C: Alternative Fiber Data Sheets

Dramix®

BEKAERT
better together

Data Sheet

DRAMIX® 3D

Dramix® 3D is the reference in steel fibre reinforcement. Combining high performance, durability and ease-of-use, 3D provides you with a time-saving and cost-efficient solution for most common applications.

> original anchorage
> standard tensile strength

Dramix® 3D is a cost efficient solution for
> flooring
> tunnel applications
> precast
> residential applications

Bekaert supplies all of the support you need for your project. We help you determine the most suitable fibre types, calculate optimal dosages, select the right concrete quality. Contact your local support.

Go to www.bekaert.com/dosingdramix for our recommendations on handling, dosing and mixing.

Modifications reserved.
All details describe our products in general form only.
For detailed information, product specifications available on request.

PERFORMANCE

Material properties

Tensile strength: $R_{m,nom}$: 1.345 N/mm²
Tolerances: ± 7,5% Avg
Young's Modulus: ± 200.000 N/mm²

Geometry

Fibre family 3D
Length (l) 30 mm
Diameter (d) 0,55 mm
Aspect ratio (l/d) 55 (80, 65, 55, 45)

Fibre network

13,4 km per m³ (for 25 kg/m³)
17.054 fibres/kg

Dramix® range

	5D	4D	3D
Tensile strength			
Wire ductility			
Anchorage strength			

PRODUCT CERTIFICATES

CE 0749-CPD EN 14889-1 — Conforms to ASTM A820

Dramix® is certified for structural use according to EN 14889-1 (system '1'). Detailed information is available on request.

SYSTEM CERTIFICATES

All Dramix® plants are ISO 9001 and ISO 14001 certified.

PACKAGING

BAGS 20 kg BIG BAG 800-1100 kg

STORAGE

KEEP DRY NO STACKING

PRODUCT DATA SHEET

Product Code	Issue Date	Rev. Level	Rev. Date
S-207 Chopped Steel Fiber	08-Jan-95	08	28-Sep-05

PROPERTY		**SPECIFICATION**	**TEST METHOD**
Sieve Analysis:	No. 14	0 – 1.0%	WI900-2
	No. 20	0 – 6%	
	No. 40	10 – 25%	
	No. 70	15 – 35%	
	No. 100	5 – 20%	
	Pan	30 – 55%	
Tap Volume:		85 – 120 cc/100g	WI900-1
Acetone Extract:		0.04% max	WI900-4
Chemical Analysis:	C	0.13% max.	COA
	Mn	1.25% max.	
	P	0.04% max.	
	S	0.05% max.	
	Si	0.17% max.	

PACKAGING

Multi-ply paper bags, available in 50 lbs. or 25 kgs. per bag, palletized and stretchwrapped.

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Appendix D: Impact Resistance Data

Modified ACI Drop-Weight Impact Test for Concrete											
Material: PAC-5				Material: Lock-Tuf				Material: Ductal			
Specimen #	Thickness: (mm)	First Crack	Ultimate Rupture	Specimen #	Thickness: (mm)	First Crack	Ultimate Rupture	Specimen #	Thickness: (mm)	First Crack	Ultimate Rupture
1	52.5			1	50.3	25	162	1	51.1	229	515
2	52.1	2	2	2	50.4	15	72	2	49.9	6	100
3	54.0	2	2	3	50.6	12	68	3	50.2	36	137
4	50.5	2	2	4	50.4	9	411	4	50.5	8	24
5	49.3	2	2	5	50.4	41	200	5	51.1	7	75
6	52.1	1	1	6	50.3	8	114	6	49.9	17	43
7	49.9	2	2	7	50.2	14	725	7	50.5	57	148
8	51.9	3	4	8	50.2	50	725	8	50.7	19	600
9	52.6	3	3	9	49.9	75	725	9	50	15	122
10	51.5	2	2	10	50	2	65	10	50.2	13	131
11	53	5	6	11	50.1	2	29	11	50.3	19	208
12	50.9	3	3	12	49.8	5	81	12	50	200	725
13	50.7	1	2	13	50.5	12	725	13	50.3	200	688
14	52.1	2	3	14	50.5	16	725	14	49.9	17	35
15	51.6	2	2	15	50.3	23	725	15	50	2	38
16	52.2	2	2	16	50.4	35	725	16	49.6	25	725
17	51.8	3	4	17	50.5	10	97	17	51.2	75	379
18	52.5	2	2	18	50.7	20	725	18	50.2	8	114
19	51.2	3	5	19	50.4	5	209	19	50.6	12	109
20	51.5	1	1	20	50.3	3	246	20	50	22	624
21	52.8	2	2	21	50.4	4	39	21	49.8	200	725
22	50.9	1	2	22	50.3	25	725	22	50.2	65	400
23	51.8	1	2	23	50.1	200	725	23	50.1	5	63
24	50.2	3	5	24	50.3	8	425	24	50.7	24	525
25	51.5	2	2	25	50.5	7	72	25	50.5	7	98
26	52.1	2	3	26	50.4	5	94	26	50	6	96
27	51	2	2	27	50.4	3	41	27	51.5	25	425
28	51.2	1	1	28	50	200	725	28	50.2	20	90
29	51.7	2	2	29	50.9	4	234	29	50.1	4	52
30	51.5			30	50.5	25	37	30	50.2	2	22
FAILURE CRACK					No fracture or ultimate rupture up to that point and was not close if no final number is present.						

Abbreviations

Term	Definition
ACI	American Concrete Institute
CIP	Cast-in-place
CW	Civil Works
EM	Engineer Manual
ERDC	Engineer Research and Development Center
FC	First crack
FHWA	Federal Highway Administration
GFRP	Glass-fiber-reinforced polymer
HRWRA	High-range water-reducing admixture
LT	Lock-Tuf
LVDT	Linear variable differentiated transformer
ME	Military Engineering
MOR	Modulus of rupture
MVR	United States Army Corps of Engineers Rock Island District
NavSys	Navigation Systems Research Program
o.c.	On center
RARG	Research area review group
RDEM	Relative dynamic elastic modulus
REMR	Repair, Evaluation, Maintenance, and Rehabilitation
SR	Special Report
TR	Technical Report
UHPC	Ultra-High-Performance Concrete
UR	Ultimate rupture
USACE	United States Army Corps of Engineers
w/cm	Water-to-cementitious materials ratio
WES	Waterways Experiment Station

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)* September 2023	**2. REPORT TYPE** Final	**3. DATES COVERED** *(From - To)* FY18-FY21

4. TITLE AND SUBTITLE Development and Characterization of Ultra-High-Performance Concrete for the Rehabilitation of Navigation Lock Structures	**5a. CONTRACT NUMBER** **5b. GRANT NUMBER** **5c. PROGRAM ELEMENT NUMBER**
6. AUTHOR(S) Dylan A. Scott, Stephanie G. Wood, Bradford P. Songer, Thomas Mack, Brian H. Green, Eric Hackbarth, Kirk E. Walker, and Alexander J. Tillotson	**5d. PROJECT NUMBER** 493064 **5e. TASK NUMBER** **5f. WORK UNIT NUMBER**
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) US Army Engineer Research and Development Center (ERDC) Geotechnical and Structures Laboratory (GSL) 3909 Halls Ferry Road Vicksburg, MS 39180-6199	**8. PERFORMING ORGANIZATION REPORT NUMBER** ERDC/GSL TR-23-16
9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES) Navigation Systems Research Program US Army Corps of Engineers Washington, DC 20314-1000	**10. SPONSOR/MONITOR'S ACRONYM(S)** NavSys **11. SPONSOR/MONITOR'S REPORT NUMBER(S)**

12. DISTRIBUTION / AVAILABILITY STATEMENT
Distribution Statement A. Approved for public release: distribution is unlimited.

13. SUPPLEMENTARY NOTES
"UHPC Panels"

14. ABSTRACT
This report details the history of vertical lock wall repairs and the development and laboratory characterization of an ultra-high-performance concrete (UHPC) using locally sourced materials for improved durability of lock walls subjected to impact and abrasion from navigational vessels. This UHPC, referred to as Lock-Tuf, has been designed for use in a precast environment with ambient curing methods and serves as a material proof-of-concept for future lock wall rehabilitations. Mechanical properties such as unconfined compressive strength, flexural response, tensile capacity, impact resistance, and abrasion resistance have been quantified experimentally.

15. SUBJECT TERMS
Ultra-high-performance concrete
UHPC
Abrasion
Impact
Concrete durability
Navigation
Lock wall
High strength concrete—Development
High strength concrete—Mechanical properties
Locks (Hydraulic engineering)—Maintenance & repair

16. SECURITY CLASSIFICATION OF:			**17. LIMITATION OF ABSTRACT**	**18. NUMBER OF PAGES**	**19a. NAME OF RESPONSIBLE PERSON**
a. REPORT Unclassified	**b. ABSTRACT** Unclassified	**c. THIS PAGE** Unclassified	SAR	71	**19b. TELEPHONE NUMBER** *(include area code)*