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Effects of Impure Water Sources on Early-Age Properties of Calcium Sulfoaluminate Cements for Rapid Airfield Damage Recovery

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and Christopher S. Griggs

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Abstract

In austere environments with limited access to clean water, it is advantageous to use nonpotable water for construction (i.e., mixing water for concrete.) In rapid-response situations such as rapid airfield damage recovery (RADR), the use of calcium sulfoaluminate (CSA) cements is beneficial for expedient pavement repairs because of their rapid strength gain characteristics. However, the hydration products formed by CSA cements are substantially different from those formed by ordinary portland cement and might react differently to impurities that water sources may contain. A laboratory study component investigated the application of various salts and impure sources of mixing water with commercially available CSA cement-based products. A field component studied the application of naturally occurring impure water sources for RADR. Recommendations are made for implementation of impure mixing water for RADR using commercially available flowable fill and concrete products made with CSA cement.

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Preface

This study was conducted for the Air Force Civil Engineering Center (AFCEC) Requirements, Research, Development and Acquisition Division (CXA) under project “Nonpotable Water for ADR (Airfield Damage Repair),” MIPR F4ATA75350GW01. The AFCEC-CXA technical monitor was Dr. Robert A. Diltz.

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COL Teresa A. Schlosser was the Commander of ERDC, and Dr. David W. Pittman was the Director.

1 Introduction

1.1 Objective

This report documents a laboratory experimental study and a full-scale field demonstration of the use of impure water sources (i.e., nonpotable) as mixing water for calcium sulfoaluminate (CSA) cement-based concrete mixtures used for rapid repair of infrastructure. The laboratory study effort was broadly focused on identifying the factors of primary influence on early-age properties and is documented in Chapters 3 and 4. The laboratory study was utilized for an MSCE degree thesis (Long 2019), and a portion of the laboratory data has also been published in Long et al. (2019). The field demonstration addressed the specific application of CSA cement to rapid airfield damage recovery (RADR) scenarios and is documented in Chapter 5.

Many of the applications where CSA cements are used are located on either remote field sites or disaster-devastated areas. In either instance, potable water may be logistically challenging to obtain or unavailable. The goal of this research was to understand which, if any, readily available, untreated, impure water sources would substantially affect the early-age properties of CSA cement, since changes in set time and strength development would have the greatest effect on applications where both CSA cementitious materials and impure water sources were utilized.

1.2 Purpose

The U.S. Air Force (USAF) RADR program utilizes rapid-setting repair materials containing CSA cements to quickly return damaged airfields to service. Since the USAF often operates in austere environments where potable water is scarce, it is critical to determine what types of nonpotable water can be used as mixing water for these rapid-setting materials.

To understand what may be present in untreated water sources that could affect the early-age properties of concrete, a literature review was conducted to assess the important reactions that take place in the production of CSA cement, hydration of CSA cement, and modification of CSA cement-based concretes with admixtures or heavy-metal contaminants. A literature review was also conducted to assess the concentration of impurities

typically found in untreated water (Chapter 2). Since little has been written about the effects of using impure mixing water to produce CSA cement-based concrete, historical studies on the effects of impure water sources on ordinary portland cement (OPC) were also reviewed.

The information gathered from the literature review was used to determine which impure water sources were the most important to test and whether the impure water samples had concentrations of impurities that were comparable to previously reported values. The literature review was also used as a reference when selecting high concentrations of reagents to produce synthetic solutions and ensured that the values selected were possible and could potentially be encountered in an austere environment. The water sources and concentrations of synthetic solutions were used as mixing water to prepare CSA cement-based concrete materials. Samples prepared from these concrete materials were used to test the time of set and early-age strength development of the concrete materials.

For the application of CSA cement-based concrete materials to ADR scenarios, several of the impure water sources were selected for the field demonstration. Standard ADR methods were employed, but impure water sources were substituted for normal potable mixing water. The demonstration investigated constructability issues and early-age properties (set time and strength) and conducted structural performance testing of the repairs.

2 Literature Review

2.1 Calcium sulfoaluminate (CSA) cements

CSA cements were specifically chosen for this study because this material gains strength rapidly, is commercially available, and is relatively easy to obtain. Other cementitious materials, such as portland cement, attain similar unconfined compressive strengths but over a longer period of time. Due to the different properties of these cementitious materials, it is important to assess how this material may be affected by construction practices, environmental factors, and in-service use. This report is particularly focused on how the use of locally available, untreated, impure water sources could impact the setting behavior and strength development of CSA cements.

2.1.1 History and usage of CSA cements

Unlike traditional portland cements, CSA cements contain the mineral phase ye'elimite. Ye'elimite was first synthesized by Ragozina in 1957 (Ragozina 1957; Hargis et al. 2014) and was first identified as having a composition of $C_4A_3\bar{S}$ by Fukuda in 1961 (Hargis et al. 2014; Fukuda 1961). In the United States in the 1960s, Klein (1964) patented a cementitious material that could be added to or used in lieu of portland cement and contained ye'elimite. This cementitious material was noted to compensate at least partially for shrinkage in portland cement systems and could even be used to produce self-stressing cements. By the 1970s, cements incorporating 30-70% ye'elimite were commercially available, most notably in China where their use was so prevalent that they became known as the “third cement series” (Zhang et al. 1999; Chen et al. 2012; Thomas et al. 2018; Zhang and Glasser 1999).

Today, prepackaged CSA cement-based concretes containing ye'elimite are widely available. This material typically costs three to five times more than similarly packaged conventional portland cement-based concrete mixtures and may require the use of special mixing equipment. However, its rapid strength development and predominately ettringite microstructure are advantageous to many applications. Notable applications have included the rapid repair or construction of roadways, airfields, locks, tunnels, and other concrete structures (Barna et al. 2010; Priddy et al. 2016b; Priddy et al. 2007; Kurtz and Constantiner 2004; Taylor 1997; Oren et al. 2014). CSA-based cements have also

been used to stabilize contaminated soils (Howard and Bilberry 2017), and researchers are investigating the ability of the cementitious matrix to bind to and immobilize heavy metals (Berger et al. 2011; Wu et al. 2014; Albino et al. 1996; Cau Dit Coumes et al. 2009; Peysson et al. 2005; Berardi et al. 1997).

CSA cement-based concretes are approximately 1-2 pH lower than OPC concrete (Taylor 1997). This has led to research about the corrosion resistance of steel reinforced CSA cement-based concrete (Glasser and Zhang 2001). Research indicates that CSA cements may offer good corrosion resistance, but the mechanics behind this corrosion resistance are still being investigated.

Despite the needle-like porous structure of CSA cements, CSA concretes have been noted to be resistant to freeze-thaw damage and chemical attack from seawater, sulfates, chlorides, and magnesium and ammonium salts (Juenger et al. 2011; Hooton et al. 2016).

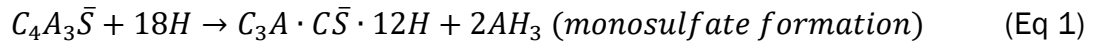
2.1.2 Production of CSA cements

The production process of CSA cement is similar to that of portland cement. The cement is produced when bauxite, red clay, and other alumina-rich materials are combined with limestone, clay, and gypsum. These raw materials are then fired at approximately 1,325°C instead of the 1,450°C conventionally used for portland cement (Zhang et al. 1999; Bullerjahn et al. 2015). Studies have also reported the successful use of alumina- and calcium-rich waste materials such as slag, fly ash, marble waste sludge, and incinerated municipal waste as additives in the production of CSA cements (Hargis et al. 2014; Taylor 1997; El-Alfi and Gado 2016). CSA cement requires less energy to grind than portland cement, and it is common for 20% calcium sulfate to be interground with the CSA cement clinker (Wu et al. 2013; Juenger et al. 2011; Valenti et al. 2012; Burris et al. 2015; Mehta 1973). The cement is typically ground to a fineness comparable to portland cement. However, coarser grinds are slower to react and, therefore, are sometimes used in shrinkage compensating or expansive CSA additives (Taylor 1997).

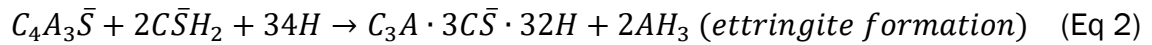
The calcium sulfate used to produce CSA cement most commonly comes from gypsum, bassanite, or anhydrite, but it may also come from industrial sources such as phosphogypsum and flue gas desulfurization (Taylor 1997; Beretka et al. 1993; Da Costa et al. 2016; Chen and Juenger 2012). Alkalis, lime, or calcium hydroxide is sometimes added to the cement as an accelerator.

2.1.3 Hydration kinetics of CSA cements

When combined with water, pure ye'elimite reacts to form monosulfate (Equation 1). This process is slow and has a dormant period of several hours. The microstructure that forms is plate-like, contributes little to strength development, and does not cause expansion (Chen et al. 2012; Juenger et al. 2011; Valenti et al. 2012).

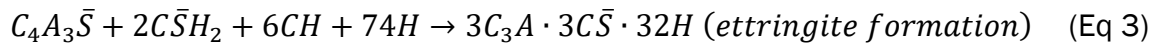


However, if calcium sulfate is present when ye'elimite reacts with water, ettringite is formed (Equation 2). Ettringite forms rapidly until the reaction consumes the available calcium sulfate, at which time monosulfate forms, as indicated below by Equation 2. If more calcium sulfate becomes available, the monosulfate may convert into ettringite (Taylor 1997; Juenger et al. 2011; Burris and Kurtis 2018).



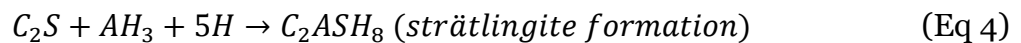
Unless measures are taken to delay ettringite formation, ettringite is formed when the concrete is in a plastic state and is generally believed to be dimensionally stable. Ettringite is the primary strength-bearing phase in CSA cements and is therefore preferable to monosulfate. As a result, calcium sulfate is typically interground during the production of CSA cement. In China, CSA cements are classified by gypsum content relative to the mass of CSA cement. In this classification system, 20% gypsum is considered to be the divide between dimensionally stable and expansive CSA cements (Juenger et al. 2011; Mehta 1969, 1973, 2006; Mehta and Klein 1965).

If calcium sulfate and calcium hydroxide are both present, ettringite forms, as described by Equation 3. The calcium hydroxide accelerates the formation of ettringite. Calcium hydroxide may be added at the time that the cement is manufactured or may be present as a by-product of belite or conventional portland cement (Chen et al. 2012; Juenger et al. 2011; Valenti et al. 2012). Since calcium hydroxide is a common hydration product of portland cement, this reaction is critical to the properties of shrinkage-compensating and self-stressing concretes. Calcium hydroxide is also common in CSA-based cements used for repair applications to accelerate rapid strength development (Taylor 1997; Juenger et al. 2011).



An example of calcium's and other alkali salts' being used to accelerate the hardening of CSA-based cements in the construction of mine walls was described by Long et al. (1987). For this application, a grout had to be pumped for a long distance, but once in place it needed to gain strength rapidly to further progress in the tunnel. As a result, two separate slurries were pumped. One contained ye'elimite and water, while the other contained calcium sulfate in the form of anhydrite and alkali salts suspended in a bentonite solution. The two resulting slurries were noted to remain dormant for very long periods of time and harden rapidly when mixed (Taylor 1997; Long et al. 1987).

Ye'elimite reacts much more readily and quickly than other mineral phases in CSA cements such as belite, calcium aluminoferrite, or calcium aluminate. When present, these other mineral phases may hydrate to form strätlingite, calcium silicate hydrate, or calcium aluminate hydrate, depending on what minerals are present in the CSA cement. Belite has a much lower rate of hydration than ye'elimite and tends to form strätlingite if enough AH_3 is present (Equation 4) (Cau Dit Coumes et al. 2017b). Even though this reaction is slower, CSA-belite (CSAB) cements continue to gain strength over time, attaining compressive strengths equal to or greater than conventional CSA cements.



2.1.4 Effects of admixtures and impurities on the early-age properties and microstructural development of CSA cements

Research has noted that the early-age strength development of CSA cements has been retarded by chloride salts, phosphates, and fluorides (Taylor 1997; Oren et al. 2014; Zajac et al. 2016; Palou and Majling 1996), while early-age strength development has been accelerated by calcium sulfate, calcium hydroxide, and alkali salts (Chen et al. 2012; Taylor 1997; Juenger et al. 2011; Valenti et al. 2012; Burris et al. 2015; Palou and Majling 1996). Other substances that retarded setting time included citric acid, borax, and hydroxylic organic compounds including sugar, tartaric acid, and gluconic acid (Taylor 1997; Burris and Kurtis 2018; Zajac et al. 2016). Lithium nitrate and lithium hydroxide were reported to accelerate strength development in CSA cements (Cau Dit Coumes et al. 2017a, 2017b).

Citric, carboxylic, and tartaric acids are believed to retard the setting time of CSA cements by binding to aqueous Ca^{2+} and Al^{3+} ions and slowing the nucleation and growth of phases such as ettringite (Taylor 1997; Burris and Kurtis 2018). Borax is believed to retard strength development because it goes into solution rapidly, raises the pH, and slows the dissolution of mineral phases such as ye'elimite (Zajac et al. 2016).

Palou and Majling (1996) noted that AlCl_3 and CaCl_2 retarded the formation of hydration products in CSA cements while Na_2SO_4 and $\text{Al}_2(\text{SO}_4)_3$ accelerated the formation of hydration products. Lan and Glasser (1996) reported that calcium chloride greatly retarded the heat generated during the hydration of CSA cements containing substantial quantities of belite.

Additionally, researchers have found that the early-age properties of CSA cements are influenced by salts found in the cement or added during mixing. Burris and Kurtis (2018) reported that CSA cements with a calcium sulfate content greater than 20% had a substantially lower strength at all ages studied when citric acid was used as a retarder. This effect was not observed in CSA cements with a calcium sulfate content less than 20%.

In general, lower sulfate contents and the formation of AFm (Al_2O_3 - Fe_2O_3 -mono) phases have been noted to correspond with increased set times and retarded strength development. The AFm phases that have been noted to form include Friedel's salt ($\text{C}_3\text{A}\cdot\text{CaCl}_2\cdot 10\text{H}_2\text{O}$), Kuzel's salt ($\text{C}_4\text{F}\cdot\text{H}_{12}$), strätlingite (C_2ASH_8), and U-phase ($\text{C}_4\text{Ao}_{0.9}\text{No}_{0.5}\text{S}_{1.1}\text{H}_{16}$).

Additionally, high concentrations of CaCO_3 in water mixed with CSA cements have been noted to promote the formation of $\text{C}_6\text{AC}\cdot\text{H}_{32}$. The resulting microstructure was noted to contain large, hollow, tubular ettringite crystals (Taylor 1997).

Zajac et al. (2016) observed that a solution made using a CSA cement and borax had a lower pH than anticipated and hypothesized that since the solution had high concentrations of Na_2O , Al_2O_3 , and SO_3 , either U-phase or Dawsonite had precipitated out.

2.2 Impure water sources

Untreated, "impure" water sources are noted to cover approximately 70% of the Earth's surface, which does not include the water present in underground aquifers (USGS n.d.). These water sources contain

impurities that are a result of both natural processes (such as dissolution of geologic structures) and land development (such as agricultural run-off, petroleum spillage, pesticides, sulfates, and other industrial waste).

From an environmental perspective, the salinity of surface water has been noted to have a profound effect on the local ecology. Two of the most common sources of salinity, chloride salts and phosphate salts, are also noted to affect the setting behavior and microstructural development of CSA cements. Other impurities, such as silt and iron, are commonly present in quantities large enough to change the appearance of the mixing water.

2.2.1 Effects of impurities on the early-age properties of ordinary portland cement (OPC)

Since there is little information on how CSA cements will react when mixed using impure water, this section focuses on historical research examining the effects of impure water sources on OPC and the different types of impure water that may be encountered in a remote location or disaster-devastated region where the use of impure water in the production of CSA cement-based concrete would be most advantageous.

In 1924, Abrams measured the effects of 52 different water sources on the setting time, consistency, and unconfined compressive strength development of OPC concrete and mortars. After running approximately 6,000 tests, Abrams (1924) concluded the following.

In spite of the wide variation in the origin and type of the waters used, and contrary to accepted opinion, most of the samples gave good results in concrete. This result seems to be due to the fact that the quantity of injurious impurities present is quite small. The following samples gave concrete strengths below the strength ratio of 85 per cent which was considered the lower limit for acceptable mixing waters: Acid waters, lime soak from tannery, refuse from paint factory, mineral water from Colorado, and waters containing over 5 per cent of common salt.

Waters specifically noted by Abrams to affect OPC concrete included a 10% reduction in strength when the sulfate content approached 1% of the mixing solution; a 23-35% reduction in strength when water from the Great Salt Lake, which contained approximately 20% sodium chloride,

was used; and a 20% reduction in strength when carbonated mineral water from Colorado was used as the mixing water, although most other sources of mineral water performed well (1924).

Abrams also noted that concrete produced using sanitary sewage attained 83-85% of the strength of the control if wet cured and 92 to 102% of the strength if air cured. Abrams cautioned against using common salt for the purpose of lowering the freezing point of the mixing water during cold weather, demonstrating that 5% salt lowers the freezing point of water about 3.3°C but results in a decrease in unconfined compressive strength of OPC of approximately 30% (1924).

In studying the effects of seawater containing approximately 3.5% salts, most of which were sodium chloride, researchers noted that OPC concrete strength increased at ages of 3 and 7 days and setting behavior was accelerated when compared to the control (Abrams 1924; Mather 1964; Etxeberria et al. 2016). However, unconfined compressive strengths at 28 days were 80 to 98% of the control. Using seawater was not recommended for use in reinforced concrete due to concerns that the ions would contribute to the corrosion of reinforcement (Abrams 1924; Etxeberria et al. 2016).

Greywater is a waste product of showers, washing machines, and other household sources that do not include toilets. This type of water describes 50-85% of the household wastewater around the world (USGS n.d.) and constitutes the water source available anywhere people reside. Alqam et al. (2014) noted that OPC concrete mixed and cured with greywater attained unconfined compressive strengths equal to that of concrete cured using potable water, and the unconfined compressive strength slightly increased at early ages (less than 7 days).

2.2.2 Concentrations of impurities in untreated water sources

Abrams' study (1924) helped to identify water impurities that may hinder strength development in OPC. However, the chemical reactions that take place during the hydration of CSA cements are different from those of OPC and may be affected differently. Present studies have focused on the use of admixtures or ions present in hazardous waste materials that must be contained but not on the concentrations of materials commonly found in the everyday environment. Early-age strength development in CSA cements has been shown to be retarded by

chloride salts, phosphates, and fluorides, while early-age strength development was accelerated by calcium sulfate, calcium hydroxide, and alkali salts. Since all of these substances are commonly found in natural water sources, it is important to quantify the concentration of each substance typically present in natural water sources (Table 1).

ASTM C1602 (2012) specifies that when nonpotable water is used as mixing water to produce hydraulic cement concrete, the unconfined compressive strength shall not be less than 90% of the control mixture produced using potable water, and the time of set must be no more than 1 hr less or 1 hr and 30 min more than the control. ASTM C1602 also specifies optional chemical composition limits for combined mixing water, which are referenced in Table 1. This guidance was written based on the setting properties and reaction kinetics of OPC and does not capture the rapid strength development that is typical of CSA cement.

Note that seawater has a composition very similar to what is typically used for “synthetic” seawater described in ASTM D1141 (Mather 1964; Hem 1959; ASTM 2013). This composition is typical for seawater, but the actual composition can vary greatly based on location, marine life, and depth (Hem 1959). Chlorides are present in all natural waters; however, in most cases these values are low. Chlorides are typically less concentrated in freshwater surface streams than sulfate and bicarbonate are (Hem 1959).

Table 1. Compositional range of natural water sources (mg/L).

Source	Concentration (g/L)		Concentration (mg/L)				
	Total Solids	Total Salinity	Ca	Mg	Na + K	SO ₄	Cl
Fresh Water, Dundee, MI [a]	2.7	2.4	340	100	400	1,480	65
Great Salt Lake, UT [a]	228	227	1,280	6,330	78,600	14,400	126,700
Briny Well, Okmulgee County, OK [b]	140	137	7,470	1,330	43,900	47	83,800
Briny Well, Midland, MI [c]	N/R	114	73,340	9,477	31,300	13	187,000
Industrial Waste, Endicott, NY [a]	9.0	6.1	1,220	trace	1650	1,000	2,200
Industrial Waste, Chicago, IL [a]	53	39	420	5,750	3,520	28,700	540
Seawater [b]	N/R	34	410	1,350	10,900	2,700	19,000
Dead Sea [d]	N/R	301	16,400	36,200	45,000	600	197,000
Minnesota Bog [a]	145	40	20	5	5	5	5
Mississippi River, New Orleans, LA [e]	223	94	34	8.9	13.8	25.5	10.3
ASTM C1602 [e] optional limits	50,000				600 ^f	3,000	500 ^g – 1,000

a) Abrams 1924

b) Hem 1959

c) Scott and Barker 1962

d) Nissenbaum 1975

e) ASTM C1602/1602-M 2012

f) (Na₂O +.658 K₂O)

g) 600 for pre-stress and bridge decks; 1000 for other reinforced concrete

3 Laboratory Materials and Methods

3.1 Research approach

Due to the criticality of restoring functionality to damaged infrastructure elements, only the early-age unconfined compressive strengths of the CSA cement-based materials were studied. This report assumes that after expedient repairs are made using CSA cement-based materials and supplies begin to flow into the region devastated by disaster, planned repairs using conventional materials and potable water sources could be staged as necessary to maintain functionality. As a result, this study did not attempt to identify long-term durability issues related to the use of impure mixing water in the production of CSA cement-based concrete.

Just as this report does not touch on the long-term effects of using impure mixing water sources, it also does not examine other factors that will affect early-age performance, such as curing temperature or placement methods. Both of these elements were closely controlled in this study to develop a clearer understanding of how impure water sources could affect early-age properties.

To accomplish the goal of this research of understanding the effects of impure water sources on early-age properties of CSA-based cement, a research program was undertaken in two parts. In the first part, 12 synthetic water solutions were prepared at concentrations similar to those found in nature using common chloride and sulfate salts. These solutions were prepared to demonstrate the effects of specific ions that may be present in impure water sources on the early-age strength development of a CSA cement-based concrete material. In the second part of the research program, the unconfined compressive strengths of CSA cement-based concrete cast using each of the 12 synthetic water solutions for mixing water were then compared to those obtained using each of six natural water and greywater sources and to a control de-ionized mixing water. Additionally, for the natural water and greywater sources, a second CSA cement-based material for flowable fill applications was tested. Time-of-set measurements were recorded for both CSA cement-based materials mixed using natural water and greywater sources.

Since this research program is focused on commercially available CSA materials and not those synthesized or optimized in the laboratory,

tests were conducted to quantify the cementitious content, bulk chemistry, and mineralogical composition of the CSA cement-based concrete. Similarly, the compositions of the impure natural water and greywater sources were also tested by an outside laboratory.

3.2 Composition of commercially available CSA cement-based concrete materials

Since commercially available concrete and flowable fill materials are sold pre-blended with aggregates, it is difficult to calculate a true cementitious content. For practical purposes, all material passing the sieve with 75- μ m openings (U.S. #200) was assumed to be predominately cementitious due to its size. This material is referred to in this article as the cementitious content of the product, even though it may contain small quantities of powdered admixtures or inert fines. The composition of the cementitious material was tested using X-Ray Fluorescence (XRF) and X-Ray Diffraction (XRD). Additional details and the results of this testing are as follows.

Ten sacks each of the commercially available, prepackaged CSA cement-based concrete and flowable fill materials were sampled at random from sacks produced in the same lot as those used to prepare concrete test specimens. Once the composition tests described in this section were complete, any material left over from the sack was discarded and not used to produce concrete test specimens.

Each prepackaged material was dry blended by using a single-shaft paddle mixer; a sample weighing a minimum of 300 g was then sieved over fine aggregate sieves in accordance with ASTM C136/C136M-14, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates* (2014). Two sieve analyses were conducted on each 22.7-kg sample of prepackaged material. The cementitious content of each mixture is reported in Table 2.

Table 2. Cementitious content of commercially available CSA cement-based concrete and flowable fill materials.

	% Passing 0.075-mm sieve (U.S. #200)		
Product	Mean	Std. Dev.	Coeff. of Variation
Concrete	28.0%	1.9%	6.9
Flowable fill	8.5%	2.8%	32.5

In addition to quantifying the cementitious content, the bulk chemistry of the cementitious portion of these 10 prepackaged samples was measured using XRF, and mineralogical composition was measured using XRD. The bulk chemistry of these samples was analyzed using wavelength dispersive XRF. The loss on ignition was measured in accordance with ASTM C114-18, *Standard Test Methods for Chemical Analysis of Hydraulic Cement* (2018b). The percentages of oxides for the pre-blended concrete material are reported in Table 3.

Table 3. Bulk chemistry (XRF) of commercially available CSA cement-based concrete materials.

Oxide	Concrete	Flowable Fill
	Mean (% mass $\pm$ Std. Dev.)	Mean (% mass $\pm$ Std. Dev.)
SiO ₂	14.15 $\pm$ 0.06	12.89 $\pm$ 0.68
Al ₂ O ₃	15.18 $\pm$ 0.06	18.20 $\pm$ 1.08
Fe ₂ O ₃	0.93 $\pm$ 0.01	1.81 $\pm$ 0.12
CaO	49.89 $\pm$ 0.23	46.24 $\pm$ 1.40
MgO	1.44 $\pm$ 0.01	1.40 $\pm$.12
SO ₃	13.94 $\pm$ 0.05	15.26 $\pm$ 0.90
K ₂ O	0.61 $\pm$ 0.01	0.43 $\pm$ 0.02
TiO ₂	0.65 $\pm$ 0.00	0.66 $\pm$ 0.06
Other minor oxides	0.35 $\pm$ 0.04	0.58 $\pm$ 0.05
Loss on ignition	2.66 $\pm$ 0.38	2.53 $\pm$ 0.22

The mineralogies of the samples were analyzed by using XRD. The diffraction patterns were collected by using the PC-based Windows version of X-Pert Pro Data Collector and analyzed using the Jade2010 program (Materials Data Inc.) Whole pattern fits of the samples provide semi-quantitative analysis of the material and are reported in Table 4.

The mineralogies of the two cements appear to be quite different despite similar bulk chemistry compositions. A meaningful variation in the anhydrite, bassanite, and ye'elimite results was reported for each of the ten samples and was also reported for the flowable fill material. However, this variation was much less for the concrete material.

Table 4. Mineralogy of a commercially available CSA cement-based concrete product as measured using XRD analysis.

Mineral	Notation	Concrete	Flowable Fill
		Mean (% mass $\pm$ Std. Dev.)	Mean (% mass $\pm$ Std. Dev.)
Anhydrite	$\text{C}\bar{\text{S}}$	20.9 ± 1.4	42.8 ± 12.62
Bassanite	$\text{C}\bar{\text{S}}+0.5\text{H}$ (hemihydrate)	4.5 ± 0.7	3.5 ± 2.4
Gypsum	$\text{C}\bar{\text{S}}+2\text{H}$	0.6 ± 0.6	0.5 ± 0.6
Ye'elimite	$\text{C}_4\text{A}_3\bar{\text{S}}$	25.6 ± 1.2	23.1 ± 5.3
Larnite	C_2S (belite)	40.2 ± 2.6	15.7 ± 4.4
Alite	C_3S	3.4 ± 2.6	8.3 ± 2.2
Calcite	CaCO_3	3.9 ± 2.7	3.2 ± 1.3
Quartz	S	0.9 ± 0.9	2.9 ± 3.3

3.3 Water source identification and characterization

Twelve synthetic water solutions (S) and six natural water and greywater samples (N) were studied in this program. The synthetic solutions were much simpler in composition than natural water or greywater sources, allowing for the effects of various ions to be controlled within the study.

3.3.1 Synthetic water solutions (S)

To prepare the synthetic water solutions, crystalline or powdered reagent-quality chemicals were added to deionized water. The concentrations of the compounds added to the water and the Solution ID used to refer to these solutions are provided in Table 5. The Solution ID for each solution consists of an “S” for “synthetic solution” followed by a shorthand notation indicating the cation and anion, respectively. The number at the end of the solution ID indicates the mass of the cation in the solution. A commercially available synthetic seawater without heavy metals, S(S), prepared in accordance with ASTM D1141 (2013), was also studied.

These synthetic solutions were prepared 2 to 24 hr prior to mixing to allow heat to dissipate prior to mixing. The solutions were mechanically agitated prior to mixing to ensure that any precipitate was reincorporated into the solution. The control deionized water was obtained by using a Labconco WaterPro PS polishing system. This water met ASTM D1193-03, *Standard Specification for Reagent Water* (2018a), requirements for Type I reagent water.

Table 5. Synthetic water solutions.

Solution ID	Reagent added	mg/L	Justification for concentration selected
S(CC0)	CaCl ₂	1,250	Typical concentration of calcium found in seawater [a, b].
S(CC7)	CaCl ₂	20,000	Calcium chloride concentration is comparable to brine from an oil well in Okmulgee County, OK [a]
S(CC29)	CaCl ₂	80,000	This represents the highest concentration of calcium chloride that is likely to be encountered from natural or contaminated water sources. This concentration is greater than that found in the Great Salt Lake and the Dead Sea and less than that found in a briny well in Midland, MI [c, d, e].
S(CS7)	CaSO ₄	42,280	The concentration of calcium cations is comparable to that found in S(CC7). However, the sulfate concentration is higher than that typically found in impure water sources.
S(KC7)	KCl	13,440	The concentration of potassium cations is comparable to the concentration of cations found in S(CC7), S(NC8), and S(CS7). This concentration of potassium is greater than that typically found in impure water sources, but impure waters containing this amount of potassium or more have been reported in a few locations, including Midland, MI [d].
S(MC4)	MgCl ₂ · 6H ₂ O	17,160	The concentration of chloride anions is comparable to S(CC7). The magnesium in this solution is more concentrated than that found in most water sources, but this concentration is still less than those found in water samples obtained from the Great Salt Lake, UT; Midland, MI; or the Chicago area [c, d].
S(MS4)	MgSO ₄	11,080	The concentration of magnesium cations is comparable to that of S(MC4). The sulfate concentration is elevated when compared to the concentration that is found in most impure water sources.
S(NC8)	NaCl	20,000	The concentration of sodium cations is comparable to the concentration of cations found in S(CC7), S(KC7), and S(CS7).
S(NC11)	NaCl	27,960	Typical concentration of sodium found in seawater [a, b].
S(NC50)	NaCl	127,100	The concentration of sodium chloride is comparable to that found in the Dead Sea [e]
S(NS1)	Na ₂ SO ₄	3,000	The sulfate content is near optional upper limits on sulfate content for mixing water that are specified in ASTM C1602/1602M-12, <i>Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete</i> [f]. This also corresponds to a sulfate concentration that is typical of seawater [b].
S(S)	Synthetic seawater without heavy metals ASTM D1141 (2013)		

(a) Hem 1959

(d) Scott and Barker 1962

(b) ASTM D1141 2013

(e) Nissenbaum 1975

(c) Abrams 1924

(f) ASTM D1602 2012

3.3.2 Natural water and greywater sources (N)

Six samples from natural water and greywater sources that were collected from different environments and could feasibly be used to produce concrete were considered in this study. The shorthand notation used to identify these sources throughout this report and a description of where the water was obtained are provided in Table 6. These water sources varied from fresh water sources, such as rainwater and river water, to saline sources, such as those found in the ocean or in brackish water. Greywater was obtained from a site where used shower water was stored in outdoor tanks until it could be transported to a water treatment facility. In each case, water was pumped directly from the water source into two 200-L drums, which was sufficient to complete all testing for this program.

Table 6. Shorthand notation for natural water sources used to prepare concrete samples.

Notation	Water Description
C(DI)	Deionized
N(B)	Brackish, sampled near Biloxi, MS, USA
N(G)	Greywater, sampled near Hattiesburg, MS, USA
N(MR)	Mississippi River, sampled near Vicksburg, MS, USA
N(P)	Eutrophic pond, sampled near Vicksburg, MS, USA
N(RN)	Rainwater, sampled near Vicksburg, MS, USA
N(S)	Gulf of Mexico Seawater, sampled near Biloxi, MS

These impure water samples were not treated prior to mixing the CSA cement-based concrete since using untreated water would be the most expedient and cost-effective way to obtain the mixing water necessary to produce materials for rapid repairs or soil stabilization. To ensure that the water makeup did not undergo biological changes during the testing, the water was kept at 5°C until samples were pulled from the bulk containers 24-72 hr before testing or mixing with concrete.

The composition of each of the natural water sources was characterized at a commercial water quality testing laboratory. Total dissolved solids were analyzed in accordance with Standard Method 2540C, *Standard Methods for the Examination of Water and Wastewater: Solids* (2017). Anions were analyzed by Ion Chromatography (IC) at 48 hr and at 28 days in

accordance with EPA Method 300.0, *Methods for the Determination of Inorganic Anions by Ion Chromatography* (1993). Metals were analyzed through inductively coupled plasma (ICP) in accordance with EPA SW-846 Method 6010C, *Inductively Coupled Plasma-Atomic Emission Spectrometry* (2007). Turbidity, corrosivity or pH, and total organic carbon (TOC) were analyzed in accordance with Standard Methods (SM) 2130B, *Standard Methods for the Examination of Water and Wastewater: Turbidity* (2017); SM 4500 H+B, *Standard Methods for the Examination of Water and Wastewater: pH Value* (2017); and SM 5310B, *Standard Methods for the Examination of Water and Wastewater: Total Organic Carbon* (2017), respectively. The results are reported in Table 7 using the naming convention from Table 6.

The Mississippi River composition is surprisingly similar to that reported by the USGS in 1963 (Hem 1959; Livingstone 1963). The rainwater contained more carbonates than other researchers had reported. Otherwise, the composition of the rainwater was similar to what other authors had found in the 1950s-1960s (Hem 1959; Whitehead and Feth 1964; Gambell and Fisher 1966; Junge and Werby 1958).

Table 7. Composition of natural water sources used to prepare concrete samples.

Category	Analyte	N(B)	N(G)	N(MR)	N(P)	N(RN)	N(S)	Units
Metals	Ca	47	7.2	17	15	1.6	93	mg/L
	Mg	130	2.5	8.1	67	0.13	270	mg/L
	K	56	5.9	5.8	4.9	0.22	120	mg/L
	Na	1100	100	7	21	7	2600	mg/L
General Chemistry	TOC	7.4	22	5.7	4.1	1.5	6	mg/L
	Turbidity	1.9	16	100	5.1	1	14	NTU
	Hardness as CaCO ₃	800	46	180	80	12	1600	mg/L
	TDS	3500	330	140	150	27	8100	mg/L
	pH	7.5	7.3	7.4	8.5	7.0	7.7	-
Anions	Chloride	2200	58	11	21	2.9	4600	mg/L

The seawater sample was more dilute than that typically observed for seawater. This was not unexpected, as the water was sampled from a depth of less than 1 m in the Gulf of Mexico near Biloxi, MS. This is a region where the water is shallow, and there is a large amount of freshwater from streams and runoff intermixing with the seawater. Although this may be dilute for seawater, it is a real-world scenario that

an engineer may face if using seawater as mixing water to repair critical infrastructure rapidly. The results obtained were also compared to that of a synthetic seawater, prepared at concentrations comparable to that typically found in seawater (ASTM 2013).

For the ions included in this study, the pond water sampled from Vicksburg, MS, was remarkably similar to the Minnesota Bog water sampled by Abrams in the 1920s (Abrams 1924). With the exception of being low in calcium, for the ions studied, the brackish water was comparable to the industrial wastewater sampled by Abrams in the 1920s (Abrams 1924).

3.4 CSA concrete specimen preparation

Each test age included at least four unconfined compressive strength specimens cast from at least two different batches of CSA cement-based concrete. Additional replicates were cast from batches utilizing de-ionized water, synthetic seawater, natural water, and greywater sources in order to generate more statistically robust results for the control and more complex mixing water sources.

3.4.1 Mixing procedures

All mixing took place in a temperature-controlled room that was kept at a constant $23 \pm 2.0^{\circ}\text{C}$. The dry materials were stored in this room for at least 20 hr prior to mixing to ensure that the material's temperature was at equilibrium with the room.

This study used the manufacturers' maximum recommended water content. For the concrete material, this meant that 3,550 mL of water was added per 22.7 kg of pre-blended material, which was equivalent to approximately a 0.56 water-to-cementitious material ratio. For the flowable fill material this meant that 4,725 mL of water was added per 22.7 kg of pre-blended material, which was equivalent to approximately a 2.45 water-to-cementitious ratio. It was believed that this would maximize the impact that impurities would have on the properties of the concrete and provide data on the worst-case scenario for concrete performance. A visual inspection was conducted after the material was mixed to ensure no segregation was apparent prior to casting test specimens.

3.4.2 Time-of-set measurements

Initial and final sets were measured in accordance with ASTM C403/403M-16, *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance* (2016c). To prevent the coarse aggregate from interfering with time-of-set measurements, each of the concrete mixtures was wet-sieved over a sieve with 4.75-mm openings.

3.4.3 Curing and unconfined compressive strength testing

All cylinders that were tested for unconfined compressive strength were cast, consolidated, and finished in accordance with ASTM C192/192M-16, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory* (2016a), with the exception that the cylinders were cured in an ambient 23°C air environment instead of by wet curing. This exception was made per the manufacturer's recommendations. These 75- x 150-mm cylinders were tested for unconfined compressive strength in accordance with ASTM C39/39-M, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens* (2016b).

The ends of the specimens tested at 24 and 168 hr were ground to ensure smoothness requirements were met. There was not sufficient time prior to testing to grind the ends of the specimens tested at 2 hr, and neoprene caps were used to prepare the ends for testing in accordance with ASTM C1231/C1231M-15, *Standard Practice for Use of Unbonded Caps in Determination of Compressive Strength of Hardened Cylindrical Concrete Specimens* (2015a). Strengths of some specimens at 2 hr were below the 10 MPa minimum strength requirements of ASTM C1231. A maximum of two cylinders were tested from a single batch at 2 hr and 24 hr, and a maximum of three cylinders were tested from a single batch at 168 hr. This allowed for the strength testing to take place ± 15 min at 2 hr, ± 30 min at 24 hr, and ± 1 hr at 168 hr.

4 Laboratory Results and Analysis

For the laboratory mixed and cured test specimens, the data and statistical analysis are located in Table A1 to A5 of the appendix. A one-way analysis of variation (ANOVA) was conducted and used to calculate pooled standard deviation for each of the early-age properties. Bar charts were plotted to display the 95% confidence interval after applying Bonferroni's correction. The Dunnett multiple comparison tool was used to evaluate whether the results were statistically different from those of the control (Dunnett 1964).

4.1 Early-age properties of commercially available CSA cement-based concrete prepared using synthetic solutions (S)

The synthetic solutions in this study were divided into five categories: calcium chloride solutions, sodium chloride solutions, sulfate solutions, common metal chlorides, and constituents of salinity commonly found in seawater. This approach helps to illustrate the effects of ion concentration and to directly compare the effects of different cations or anions.

4.1.1 Calcium chloride solutions

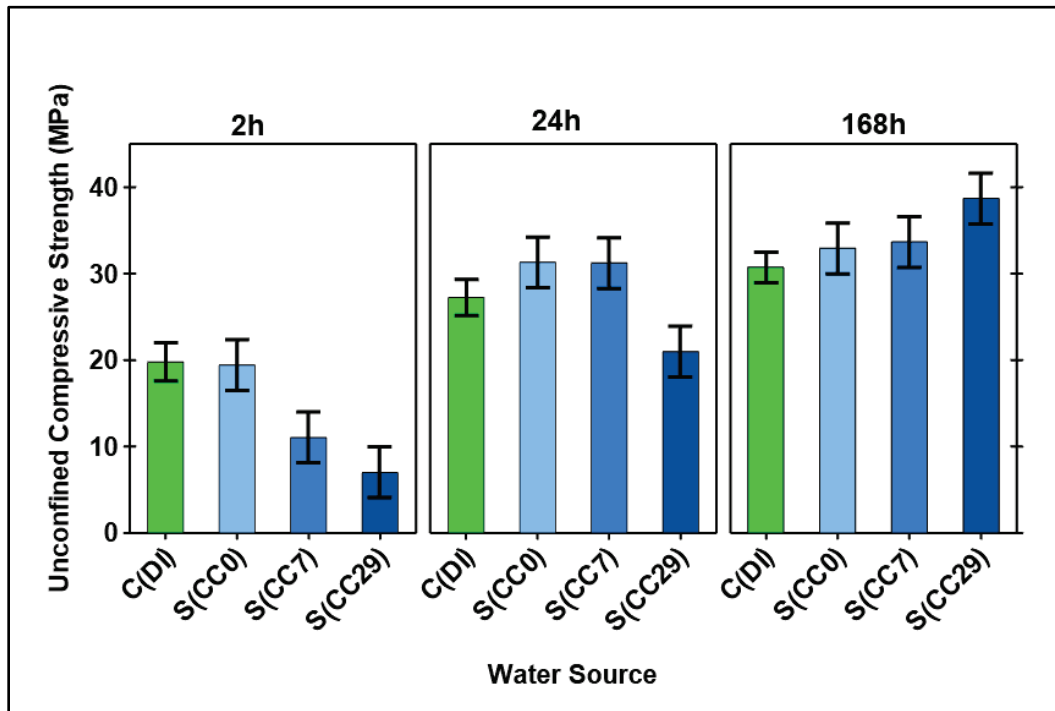
In order to study the effects of calcium chloride on early-age CSA cement-based concrete strength development, three concentrations were selected. The results showed a retardation of unconfined compressive strength development when large amounts of calcium chloride were added to the CSA cement-based concrete mixture. This effect was quite severe at high concentrations, as shown in Figure 1.

This effect lessened at later ages, where mixtures prepared using concentrated calcium chloride solutions attained unconfined compressive strengths that were statistically greater than that of the control. This behavior was noted for mixtures prepared using S(CC7) at 24 and 168 hr and for S(CC29) at 168 hr.

The retarded strength development of CSA cement-based concrete that utilized mixing water containing calcium chloride supports findings by Lan and Glasser (1996), who observed that the addition of calcium chloride to a CSA cement containing belite greatly lessened the cumulative heat of hydration of the cement at 24 hr. Lan and Glasser (1996) also noted the formation of Friedel's salt after XRD analysis and, with a microscope, they

observed that Friedel's salt appeared to coat the partially hydrated clinker grains and create a barrier between the water and the cement grains.

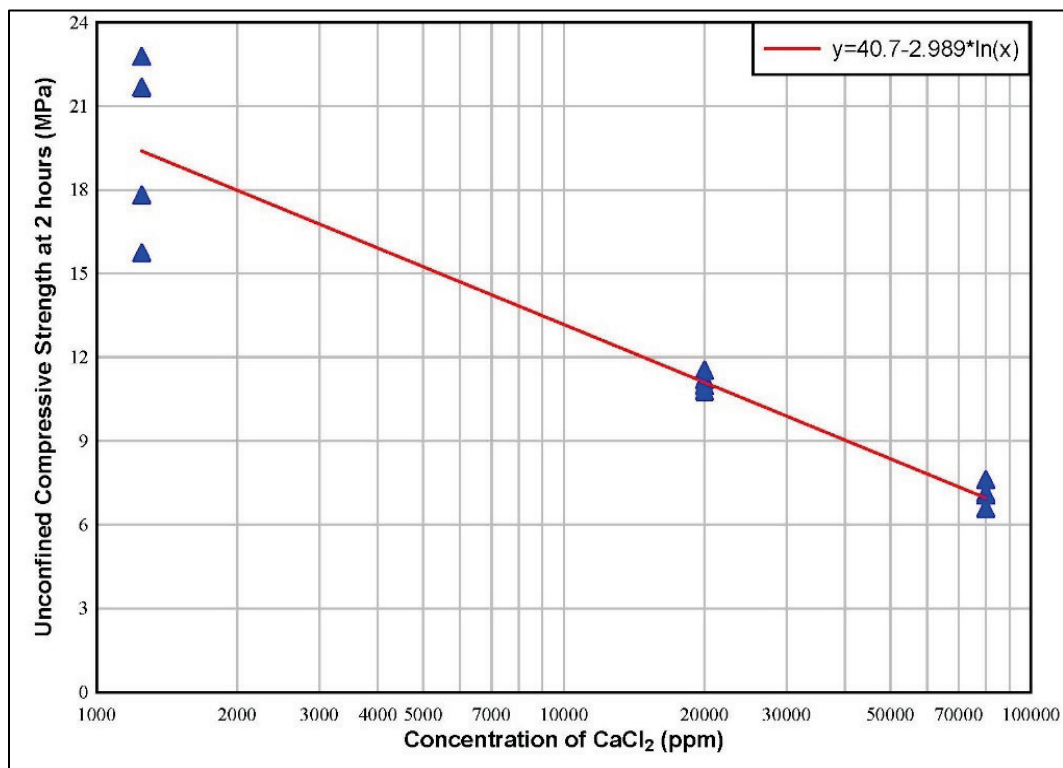
Figure 1. Unconfined compressive strength of CSA cement-based concrete when mixed using solutions containing calcium chloride at test ages of 2, 24, and 168 hr.



From a field engineering perspective, it is also important to note that calcium chloride can have a retarding effect on the strength development of CSA cement-based concrete. This is very different from OPC concretes, where calcium chloride is sometimes used as an accelerator. However, in order to have a substantial retarding effect, the source of calcium chloride must be relatively concentrated. The typical concentrations of calcium chloride found in surface freshwater and normal seawater were not great enough for a retarding effect to be observed.

Interestingly, the retardation effects noted at 2 hr showed a strong correlation to the natural log of the concentration of calcium chloride in the solution (Figure 2). It is also noteworthy that while calcium chloride is a powerful accelerator for OPC-based concrete, it clearly retards strength development in CSA cement-based concrete.

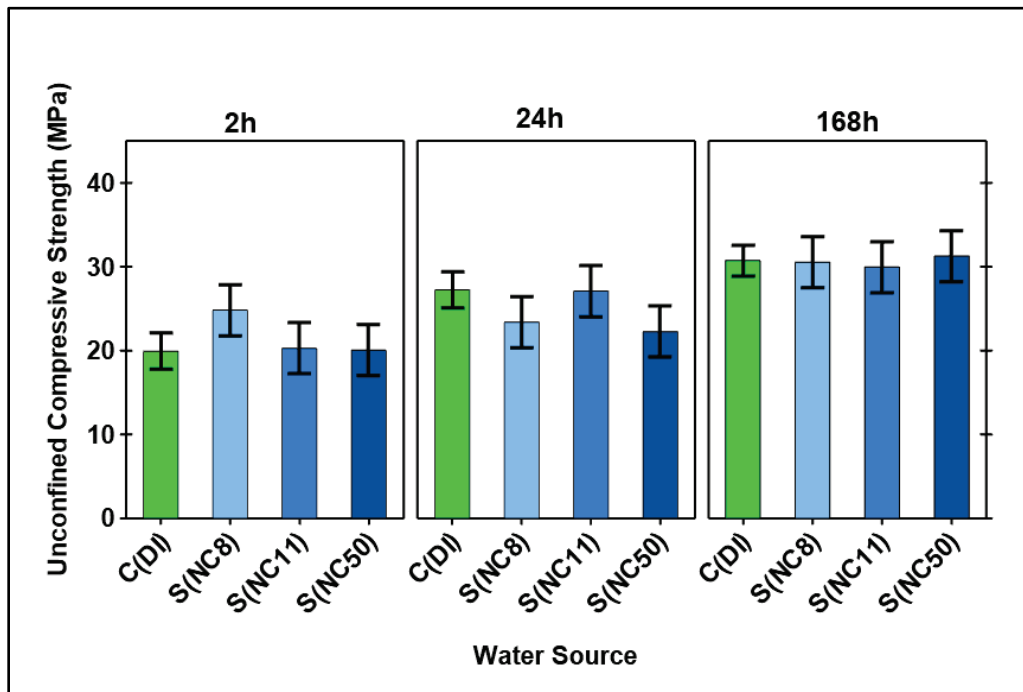
Figure 2. Effect of calcium chloride dosage on unconfined compressive strength at 2 hr.



4.1.2 Sodium chloride solutions

Sodium chloride occurs in nature at a substantially greater concentration than calcium chloride. To study the impact of sodium cations compared to calcium cations, three concentrations of sodium chloride were compared to the control. The early-age unconfined compressive strength results for these three solutions and a control are shown in Figure 3. Unlike calcium chloride, the concentration of sodium chloride in the mixing water had a negligible effect on the early-age unconfined compressive strength of concrete made using CSA cement, even at much higher concentrations.

Figure 3. Unconfined compressive strength of CSA cement-based concrete when mixed using solutions containing sodium chloride at test ages of 2, 24, and 168 hr.



The mean unconfined compressive strengths of concrete produced using mixtures S(NC8) and S(NC50) were lower than the mean of the control at 24 hr. While this strength reduction was great enough to be statistically significant, the unconfined compressive strengths of the S(NC8) and S(NC50) mixtures were statically greater than and comparable to the control, respectively, at 2 hr. Furthermore, the intermediate concentration of sodium chloride S(NC11) had an unconfined compressive strength that was comparable to the control at all ages studied. Concrete prepared using all three concentrations of sodium chloride had unconfined compressive strengths that were comparable to the control at 168 hr.

The authors do not believe that the observed deviations from the control are indicative of adverse effects on the early-age unconfined compressive strength development of CSA cement-based concrete produced using mixing waters containing sodium chloride. These concentrations include those most likely to be encountered in natural water sources.

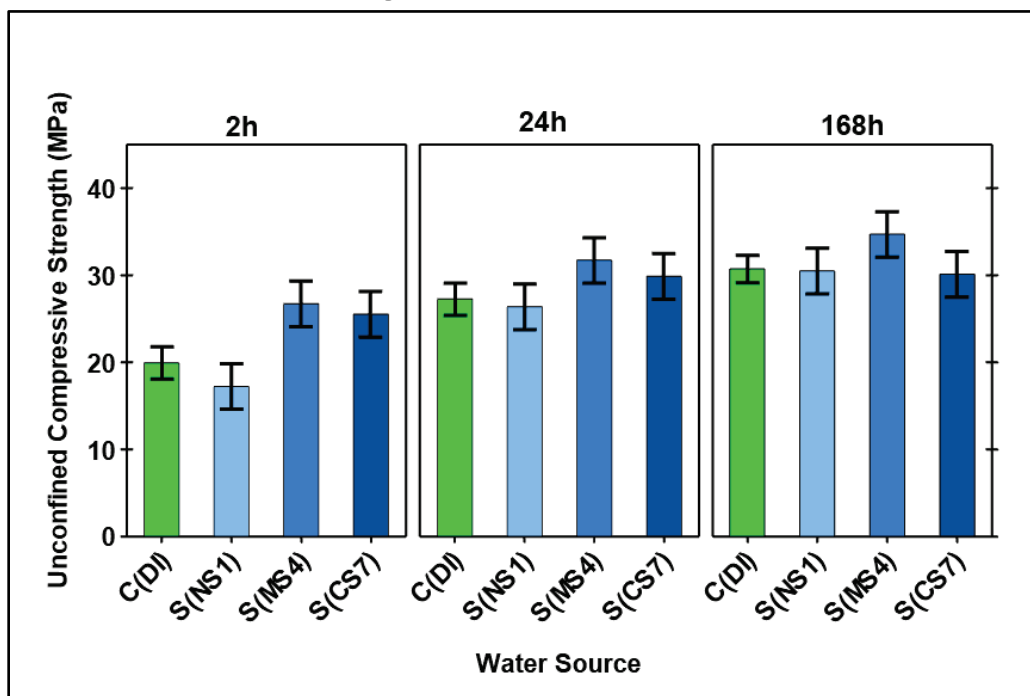
4.1.3 Sulfate solutions

Calcium sulfate is typically used to produce the CSA clinker and is interground into the CSA cement to control setting behavior. Since calcium sulfate and other forms of sulfate are commonly found in impure

water sources, the effects of sodium, magnesium, and calcium sulfate on early-age strength development were also studied (Figure 4).

The sodium sulfate S(NS1) had a negligible effect on concrete strength development, but the mixing water containing sodium sulfate was also less concentrated than other solutions containing either sodium or sulfate ions. Additional calcium sulfates S(CS7) in the mixing water accelerated strength development at 2 hr and 24 hr.

Figure 4. Unconfined compressive strength of CSA cement-based concrete when mixed using solutions containing sodium, magnesium, and calcium sulfate at test ages of 2, 24, and 168 hr.



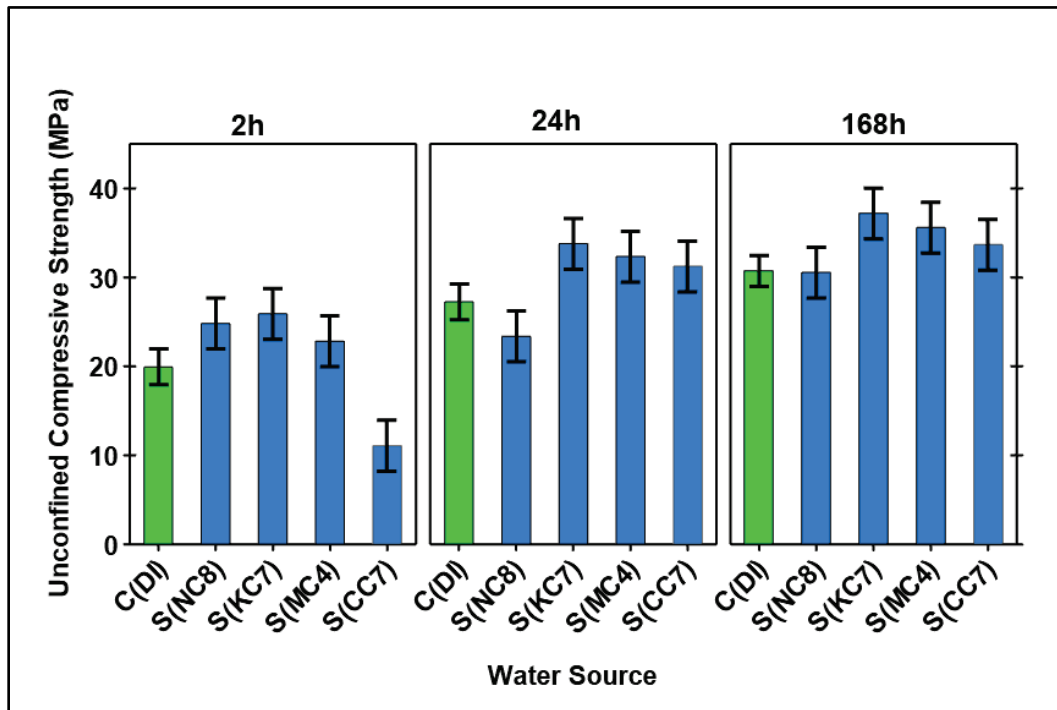
By the time the concrete had cured for 168 hr, the impact of the sodium and calcium sulfate mixing water on concrete unconfined compressive strength was negligible. Concrete produced using the magnesium sulfate solution S(MS4) had an unconfined compressive strength that was statically greater than the control at all curing times, and no statistically significant reductions in strength were observed for any of the solutions.

From a field engineering perspective, this study did not find any adverse effects on the early-age unconfined compressive strength of the CSA cement-based concrete as a result of the addition of sodium, magnesium, or calcium sulfate at the concentrations studied. These concentrations include those most likely to be encountered in natural water sources.

4.1.4 Common metal chlorides

Four of the most common cations found in impure water sources include calcium, sodium, magnesium, and potassium. In order to see whether the chloride salt that forms with each of these cations would affect the CSA cement-based concrete differently, similar concentrations of calcium, sodium, and potassium chloride were selected. A concentration of magnesium chloride that is elevated from that typically found in nature but at a lower concentration rate than the other chloride solutions was also studied, since the higher dosage rate was unlikely to occur in nature. The results of this testing are shown in Figure 5.

Figure 5. Unconfined compressive strength of CSA cement-based concrete when mixed using sodium, potassium, magnesium, and calcium chloride at test ages of 2, 24, and 168 hr.



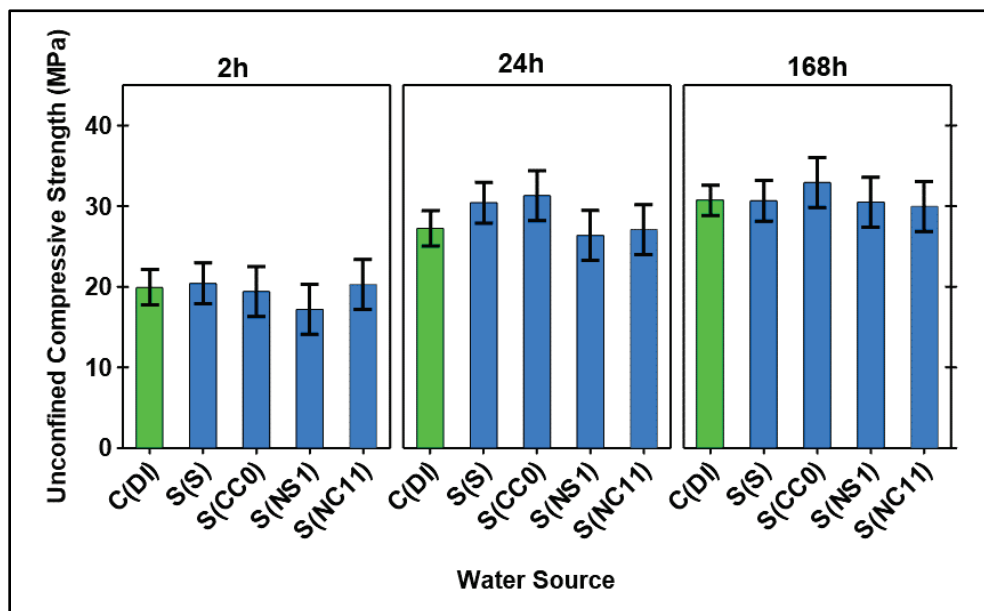
When compared to calcium chloride, the same reduction in 2-hr strength was not observed when concrete was prepared using mixing water containing sodium chloride, potassium chloride, or magnesium chloride. In fact, the mixture incorporating potassium chloride had the greatest strength of any of the synthetic solutions studied. This effect was observed at all three ages tested in this study. At the age of 168 hr, potassium chloride, magnesium chloride, and calcium chloride all had a positive effect on strength development.

From a field engineering perspective, solutions containing potassium chloride, sodium chloride, and magnesium chloride did not reduce early-age unconfined compressive strength development in CSA cement-based concrete when the solutions were prepared at concentrations comparable to those found in highly concentrated natural water sources. Calcium chloride retarded strength development after 2 hr, after which the concrete continued to gain strength, meeting or exceeding the strength of the control at 24 and 168 hr. Moderate increases in early-age unconfined compressive strength were observed when concrete was prepared using potassium and magnesium chloride mixing waters.

4.1.5 Seawater

Seawater contains many different ions and is likely to be available at remote locations where the use of CSA cement-based repair materials would be advantageous. Solutions of calcium chloride S(CCo), sodium chloride (NC11), and sodium sulfate S(NS1) were selected at concentrations near that of typical seawater. The results were then compared to those of synthetic seawater, S(S), prepared in accordance with ASTM D1141 (2013) (Figure 6).

Figure 6. Unconfined compressive strength of CSA cement-based concrete when mixed using solutions that contain concentrations of specific ions that are similar to that typically found in seawater at test ages of 2, 24, and 168 hr.



From a field engineering perspective, this study did not find any adverse effects on the early-age unconfined compressive strength of the CSA cement-based concrete as a result of the addition of calcium chloride,

sodium chloride, sodium sulfate, or synthetic seawater without heavy metals at the concentrations studied. These concentrations include those commonly associated with seawater.

4.2 Early-age properties of commercially available CSA cement-based concrete products prepared using natural water and greywater sources (N)

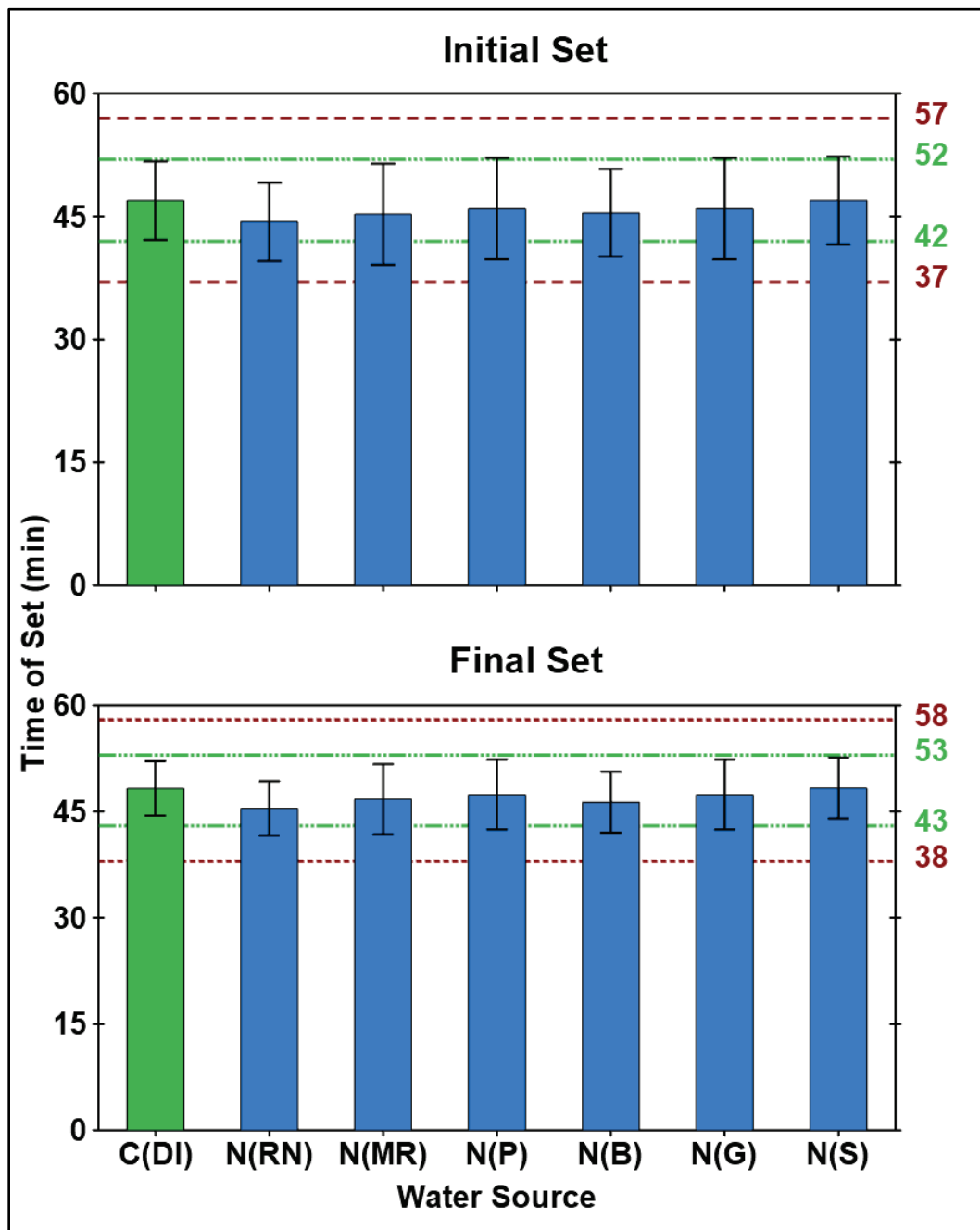
After studying the effects of simplified synthetic solutions, time-of-set and unconfined compressive strength specimens were prepared from commercially available CSA cement-based materials and mixing water obtained from natural water and greywater sources. The CSA cement-based materials were expanded to include both concrete and flowable fill.

Since the composition of each water source is complex and the CSA products have different compositions, it was important to test these materials to see whether any adverse reactions, particularly in strength development, were noticeable in the first seven days after the material was cast.

4.2.1 Time of set for CSA cement-based concrete mixed with natural water and greywater sources

Initial and final set occurred almost simultaneously in the concrete material, and the average set times were within 5 min of that of the control for each of the water sources studied. The setting behavior in Figure 7 was not statistically different from that of the control, and no negative effects on the setting time of the CSA cement-based concrete were observed when natural water or greywater sources were used for mixing water.

Figure 7. Initial and final set of CSA cement-based concrete mixed with natural water and greywater sources.



Reference lines are placed at ± 5 min (green) and ± 10 min (red) from initial and final set of control.

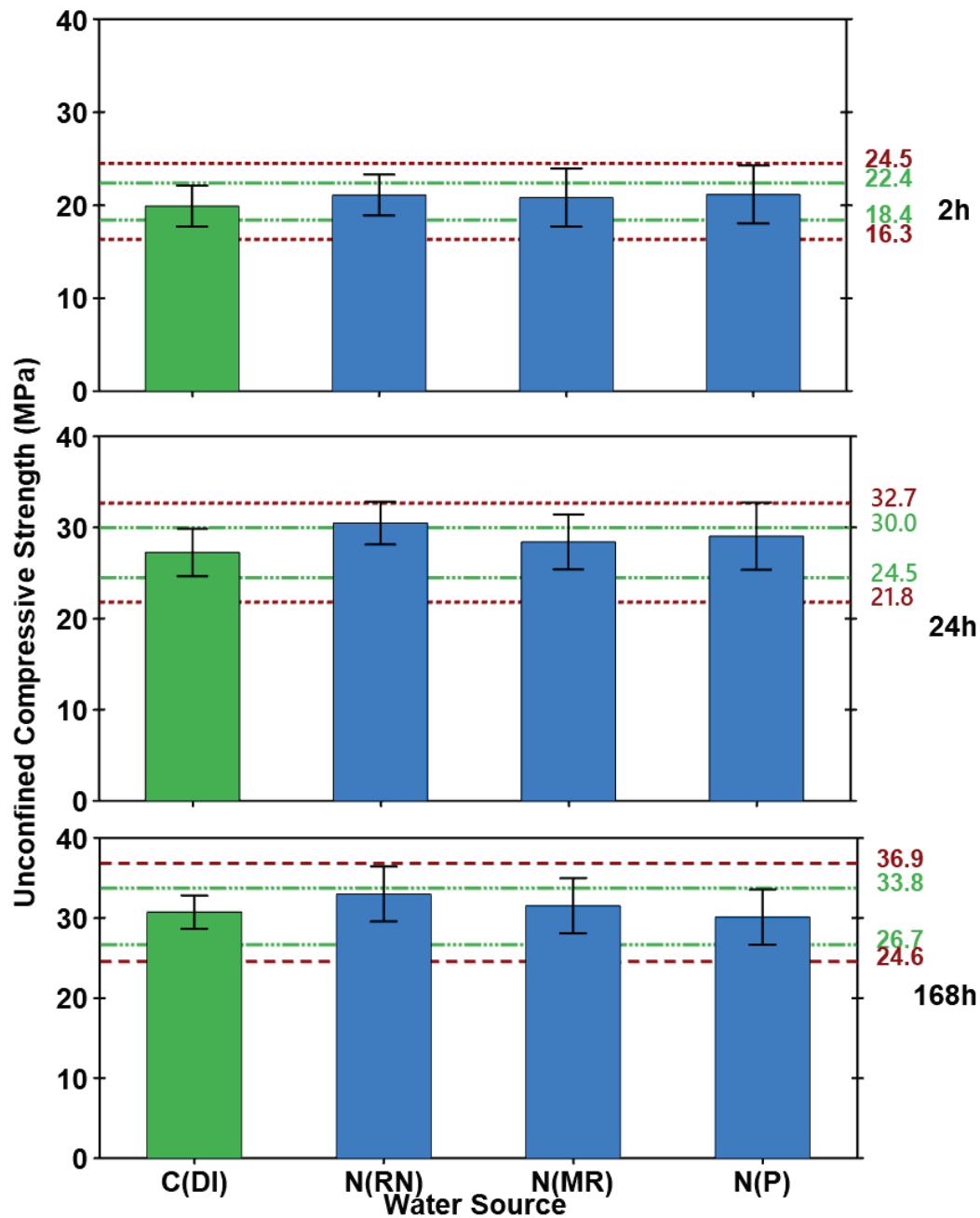
4.2.2 Early-age unconfined compressive strength for CSA cement-based concrete mixed with natural water and greywater sources

Three freshwater sources were used as mixing water for the CSA cement-based concrete: rainwater N(RN), Mississippi River water N(MR), and pond water N(P). Although these were not saline solutions, the freshwater sources contained other impurities. The freshwater sources did not appear to impact the early-age unconfined compressive strength development of the concrete (Figure 8).

Testing was also conducted using seawater N(S), brackish water N(B), and greywater N(G). Since the seawater sampled for these experiments was more dilute than typical seawater, the results are also shown compared to a commercially available synthetic seawater S(S). Despite the impurities present in these water sources, the mixing water appeared to have little impact on the early-age strength development of the concrete (Figure 9).

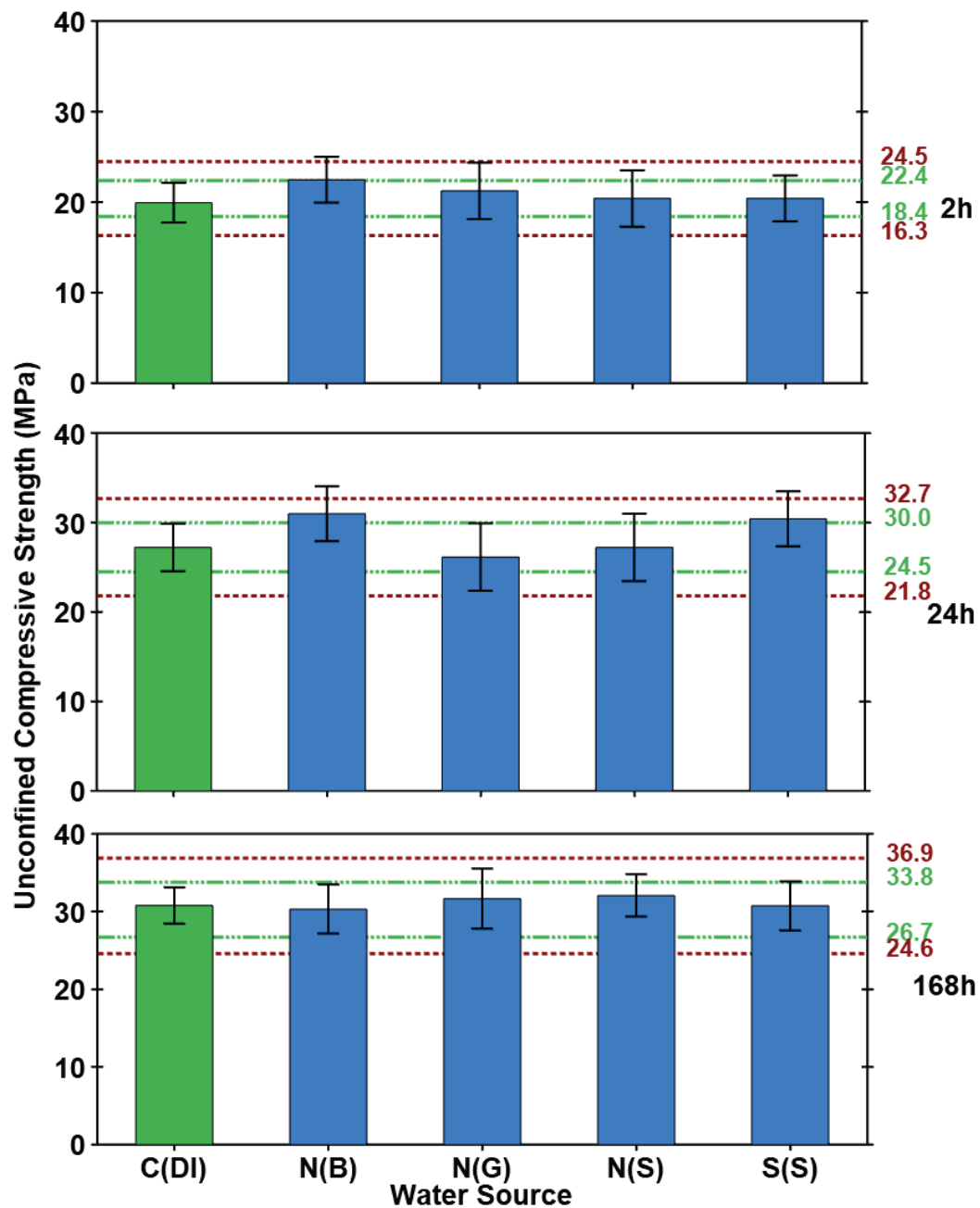
Regardless of the source of the mixing water, the average compressive strength of the CSA cement-based concrete material was at least 90% of the control, which is commonly specified in ASTM C1602 (2012). From a field engineering perspective, this study did not find any adverse effects on the early-age unconfined compressive strength of the CSA cement-based concrete as a result of using any natural water or greywater mixing water sources included in this study. These findings are also supported by the testing of synthetic solutions with similar or elevated concentrations of different constituents of salinity.

Figure 8. Unconfined compressive strength of CSA cement based concrete mixed with freshwater sources at 2, 24, and 168 hr.



Reference lines are placed at $\pm 10\%$ (green) and $\pm 20\%$ (red) from the mean unconfined compressive strength of the control.

Figure 9. Unconfined compressive strength of CSA cement-based concrete mixed with saltwater and greywater sources at 2, 24, and 168 hr.

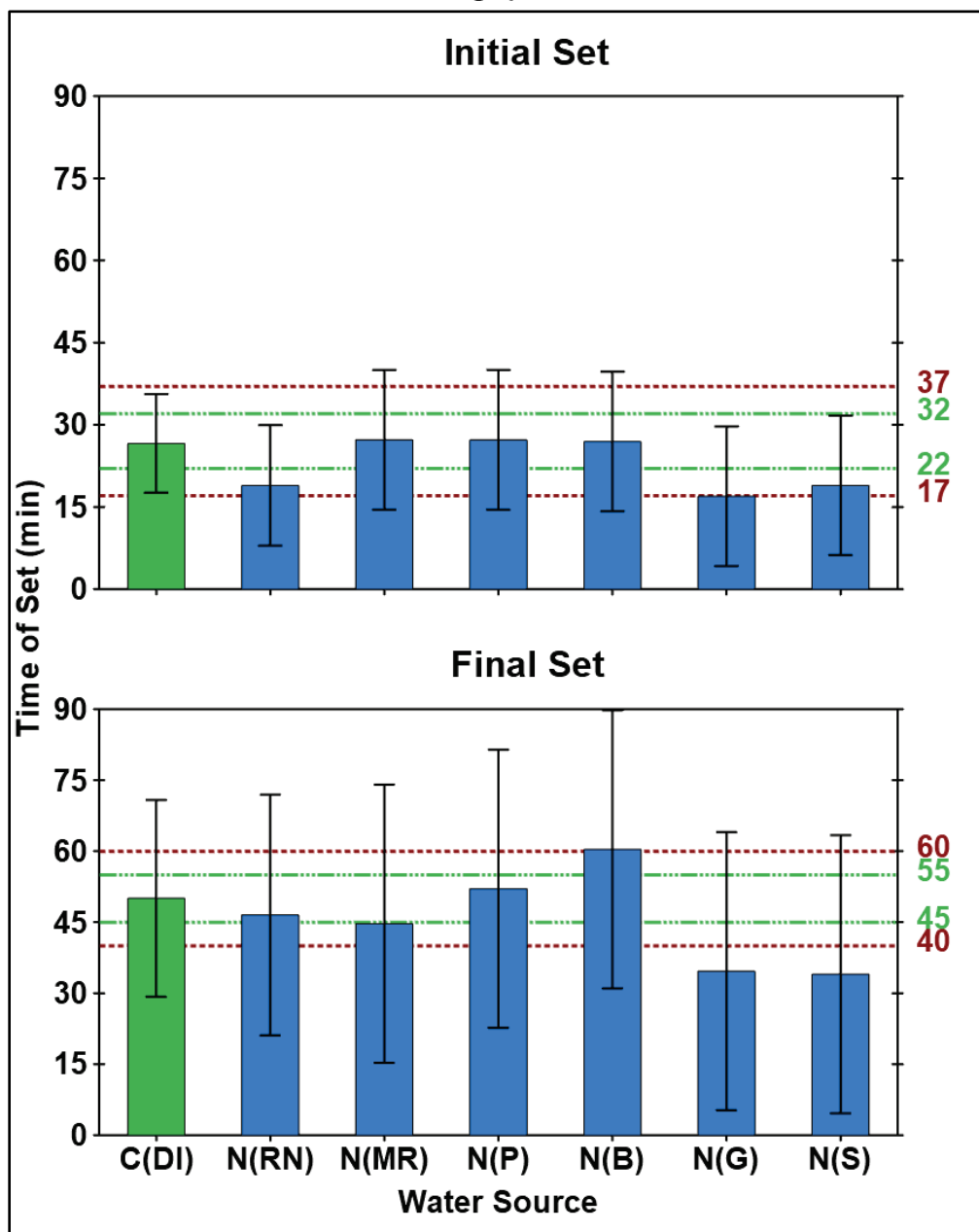


Reference lines are placed at $\pm 10\%$ (green) and $\pm 20\%$ (red) from the mean unconfined compressive strength of the control.

4.2.3 Time of set for CSA flowable fill mixed with natural water and greywater sources

The setting behavior of the flowable fill material varied much more than that of the concrete material (Figure 10). From an engineering perspective, it is difficult to determine what the setting behavior of the flowable fill material should be, even when mixed using the control water source, due to its intentionally low strength. The large variations in the cementitious content may have also contributed to this experimental variation (Section 3.2).

Figure 10. Initial and final set of CSA cement-based flowable fill mixed with natural water and greywater sources.



Reference lines are placed at ± 5 min (green) and ± 10 min (red) from initial and final set of control.

4.2.4 Early-age unconfined compressive strength for CSA flowable fill mixed with natural water and greywater sources

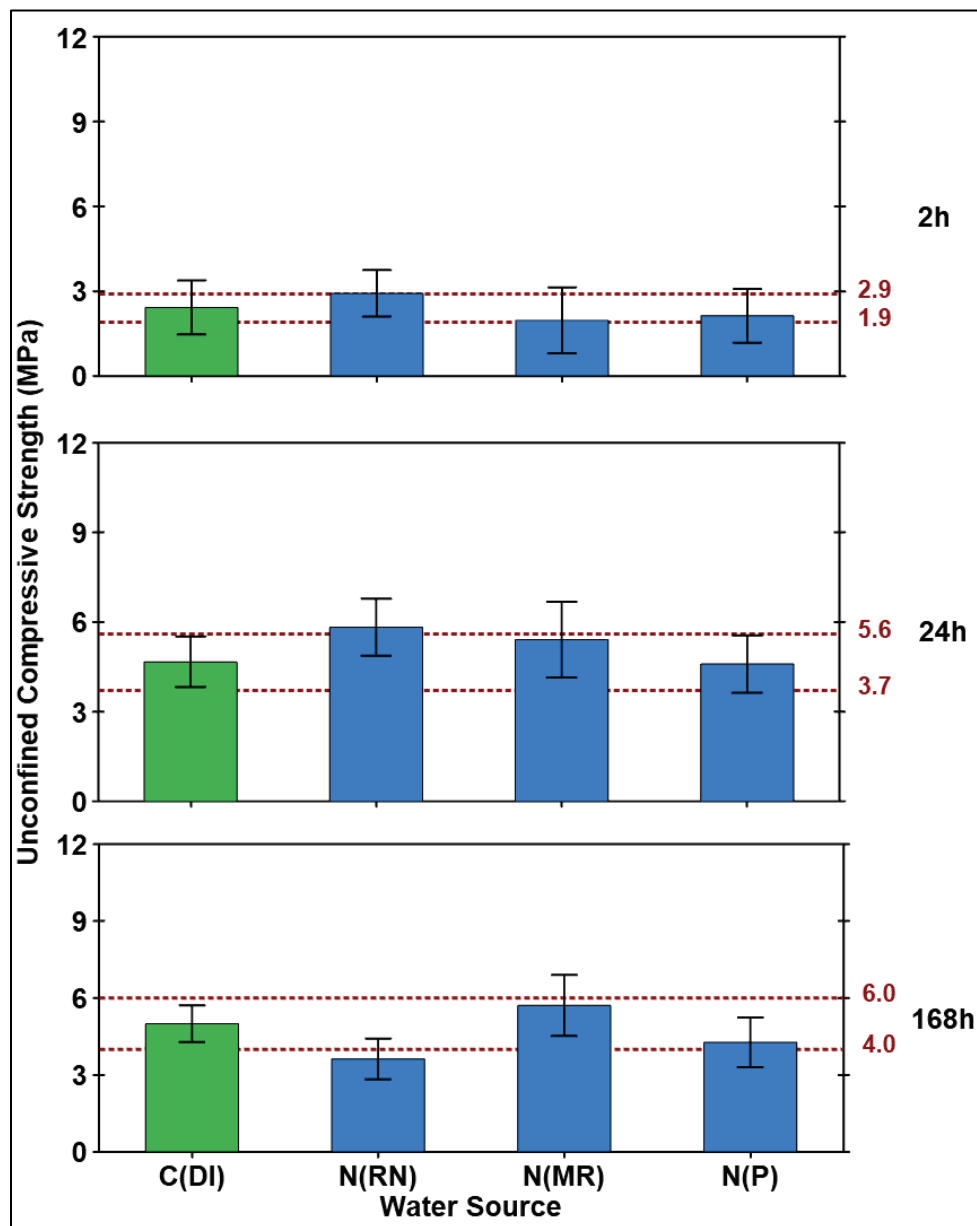
The unconfined compressive strength of the flowable fill material in Figures 11 and 12 varied substantially as a percentage of the unconfined compressive strength of the control, struggling to stay within 20% of the mean. However, the unconfined compressive strength of the flowable fill material is by definition quite low, and the actual pooled standard deviation was near 1.0 MPa.

Flowable fill samples that were cast using rainwater appeared to gain strength more rapidly than the control between 2 and 24 hr and then began to lose strength between 24 and 168 hr (Figure 11). A fourth batch was cast using the rainwater, and a similar effect was observed. A possible explanation might be that, due to the low strength of the flowable fill material and fluctuations in cement content, experimental variability in the test methods may be more pronounced, making it difficult to assess accurately variations in strength that are attributable to differences in the composition of the mixing water.

Both the seawater and the greywater produced flowable fill with unconfined compressive strengths that were statistically greater than those of samples cast using deionized water at all test ages included in this study (Figure 12). When these results are compared to initial and final set times (Figure 10), both the seawater and the greywater had an average set time 10-15 min faster than that of the control. Although the penetrometer method may not be capable of accurately measuring the time of set of the flowable fill material due to its inherently low strength, the quick set times coupled with a moderate increase in early-age compressive strength both indicate that early-age strength development could have been accelerated by the mixing water.

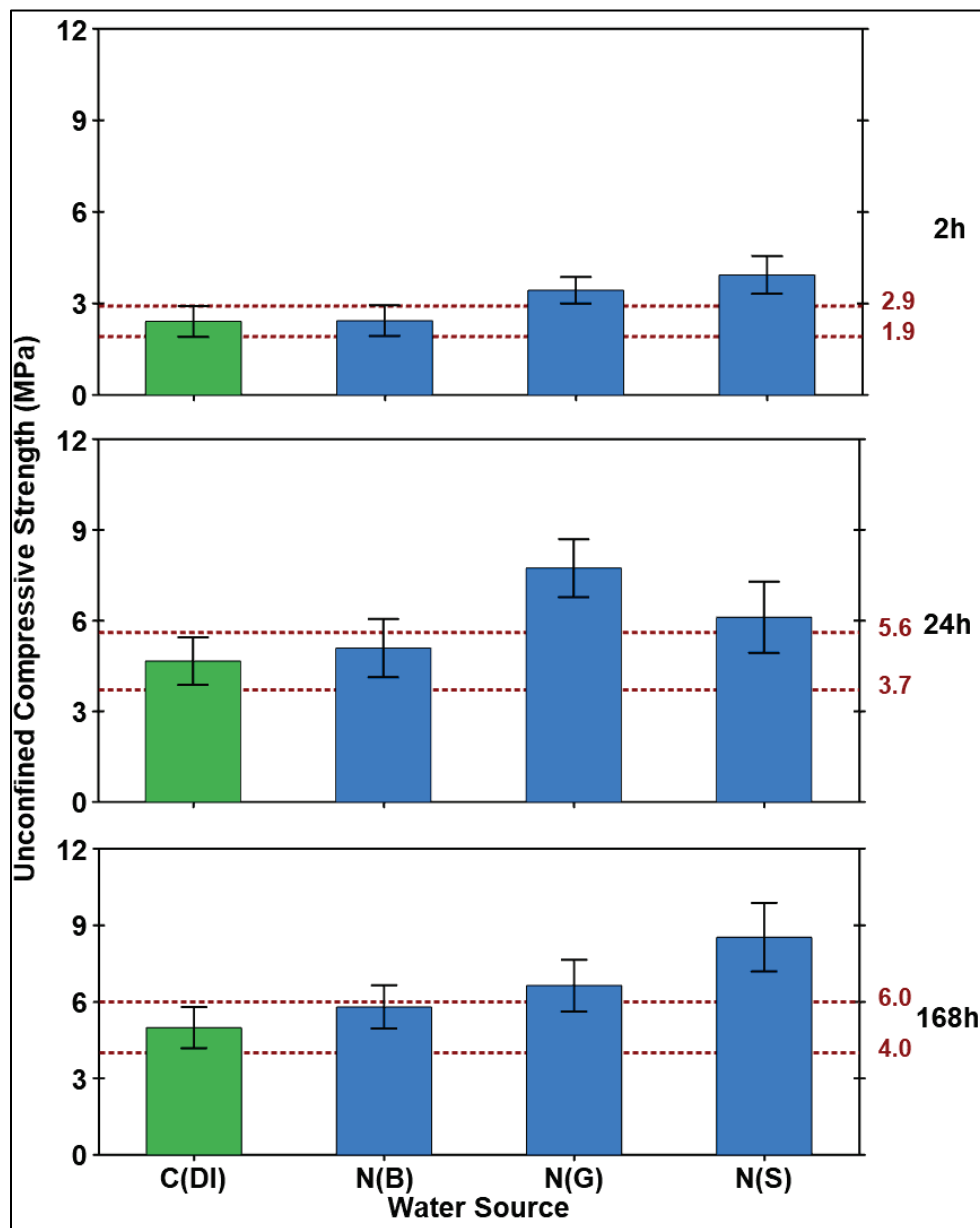
Further research on a stronger material with a similar cement chemistry to that of the flowable fill material may reveal that the strength development of the CSA cement used to produce the flowable fill is more sensitive to water impurities than is the cement used to produce the concrete material.

Figure 11. Unconfined compressive strength of CSA cement-based flowable fill mixed with fresh water sources at 2, 24, and 168 hr.



Reference lines are placed at $\pm 20\%$ (red) from the mean unconfined compressive strength of the control.

Figure 12. Unconfined compressive strength of CSA cement-based flowable fill mixed with salt water and greywater sources at 2, 24, and 168 hr.



Reference lines are placed at $\pm 20\%$ (red) from the mean unconfined compressive strength of the control.

5 Airfield Damage Repair Field Demonstration

A full-scale field demonstration was conducted to assess the use of impure mixing water sources on CSA cement materials for application in rapid airfield damage recovery (RADR). The demonstration consisted of the execution of standard sized RADR crater repairs with Buzzi Unicem flowable fill and CTS Rapid Set concrete using various impure water sources potentially available for RADR. The specific items of interest were construction aspects such as any equipment problems, material properties (set time and strength), and performance under simulated aircraft traffic loads. For the RADR application, only early-age properties were evaluated, and long-term durability was disregarded. Long-term durability of CSA concrete for airfields has been studied by Bescher and Ramseyer (2014).

5.1 Methodology

5.1.1 Crater preparation and subgrade soil characterization

To accomplish the objective, six craters were excavated in a pre-existing full-scale concrete test section remaining from a different project. The pre-existing pavement consisted of 13-in.-thick concrete slabs on top of 6 in. of limestone granular base course on natural subgrade. The subgrade soil at this location is typically classified as a lean clay (CL). The concrete slabs were 12.5 ft. wide by 15 ft. long. Each crater was in the center of a slab and was approximately 8.5 ft. by 8.5 ft. and 24 in. deep. Additional details of the crater preparation process are available in Priddy et al. (2016a).

To provide for calculation of material placement volumes and layer thicknesses, elevation measurements were made of the subgrade surface by using a rod and level. Three profiles (parallel to traffic direction) were measured as well as one cross section (perpendicular to traffic direction). Additional details of the surveying methodology are available in Priddy et al. (2016a). Horizontal dimensions of each crater were measured also.

After excavation, the in situ subgrade soil was characterized for compaction by in-place nuclear density tests in accordance with ASTM D6938 (ASTM 2015b) and for strength by Dynamic Cone Penetrometer (DCP) tests in accordance with ASTM D6951 (ASTM 2009). Three replicate measurements

were made for each test in each crater. DCP tests were analyzed to estimate California Bearing Ratio (CBR) and modulus of subgrade reaction (K-value).

Properties of the subgrade soil in each repair crater are summarized in Table 8. The CBR values ranged from 4 to 8 with an average value of 6. Similarly, the K-values ranged from 142 to 175 with an average of 163 pci. Dry density ranged from 83 to 95 lb/ft³ with an average of 89 lb/ft³. The control crater 1 had both the lowest in-place density and the lowest measured strength; however, the range of strengths between craters is relatively small and would not be expected to significantly impact the performance of these rigid pavement repairs.

Table 8. Subgrade soil in situ properties.

Crater No.	CBR (%)	K-value (pci)	Moisture (%)	Dry Density (lb/ft ³)
1: Control	4	142	11.2	82.7
2: Pond	8	164	11.7	83.2
3: River	8	162	15.9	95.1
4: Brackish	5	175	17.0	90.7
5: Seawater	6	162	19.1	87.7
6: Syn. Greywater	7	170	15.0	93.7

Notes: All data are the average of measurements made at three locations by ASTM D6951-09 or ASTM D6938-15. For DCP data, the CBR and K-values are averages of those for the depth between 0 and 18 in. from the subgrade surface.

5.1.2 Mixing water sources

The craters were filled with CSA cement repair materials produced using control and impure mixing water sources. Based on the laboratory results discussed previously, five sources of nonpotable water plus a control were identified for testing.

1. Control: municipal potable water.
2. Pond: from eutrophic pond in Vicksburg, MS.
3. River: from Mississippi River in Vicksburg, MS.
4. Brackish: from near Keesler Air Force Base, Biloxi, MS.
5. Seawater: from the Gulf of Mexico, Biloxi, MS.
6. Greywater: synthetic surfactant water to simulate soapy water discharge from showers, vehicle wash racks etc.

The control water source 1 was ordinary municipal water, since de-ionized water is not realistic for field applications. Water sources 2 through 5 correspond to those described in Section 3.3.2. Rainwater was not able to be tested in the field demonstration due to low rainfall in the preceding months. Due to concerns with bacterial contamination's creating a worker safety hazard, in lieu of using the actual greywater, a synthetic greywater was used for water source 6. This consisted of a synthetic surfactant solution (i.e., soap water) that mimics the soap content of greywater. Specifically, sodium dodecyl sulfate was used at a concentration of 20 parts per million (PPM); for comparison, shampoo would have a maximum dosage of 15 ppm. Real world greywater is more diluted than shampoo and would likely be less than 20 ppm. Bulk 500-gal samples of each water source were obtained (Figure 13a). For all materials, a volumetric concrete mixer was utilized. When switching between each mixing water source, the volumetric mixer's onboard water storage tank was drained and flushed.

Figure 13. Water sample handling.



5.1.3 Temperature measurements

To monitor changes in mixture hydration rates that might occur between the various repair craters, I-button thermocouple concrete maturity

sensors (made by Command Center, a part of The Transtec Group Inc.) were embedded into the materials to monitor temperature changes.

For each crater, I-buttons were placed on stands at both the center of the crater (equidistant from all sides) and the northeast corner at a distance of 4 in. from the north and east sides of the crater. Each stand consisted of a segment of #3 rebar driven into the subgrade to which the I-button sensors were attached at target depths (Figure 14 and Table 9).

Figure 14. Thermocouple installation.

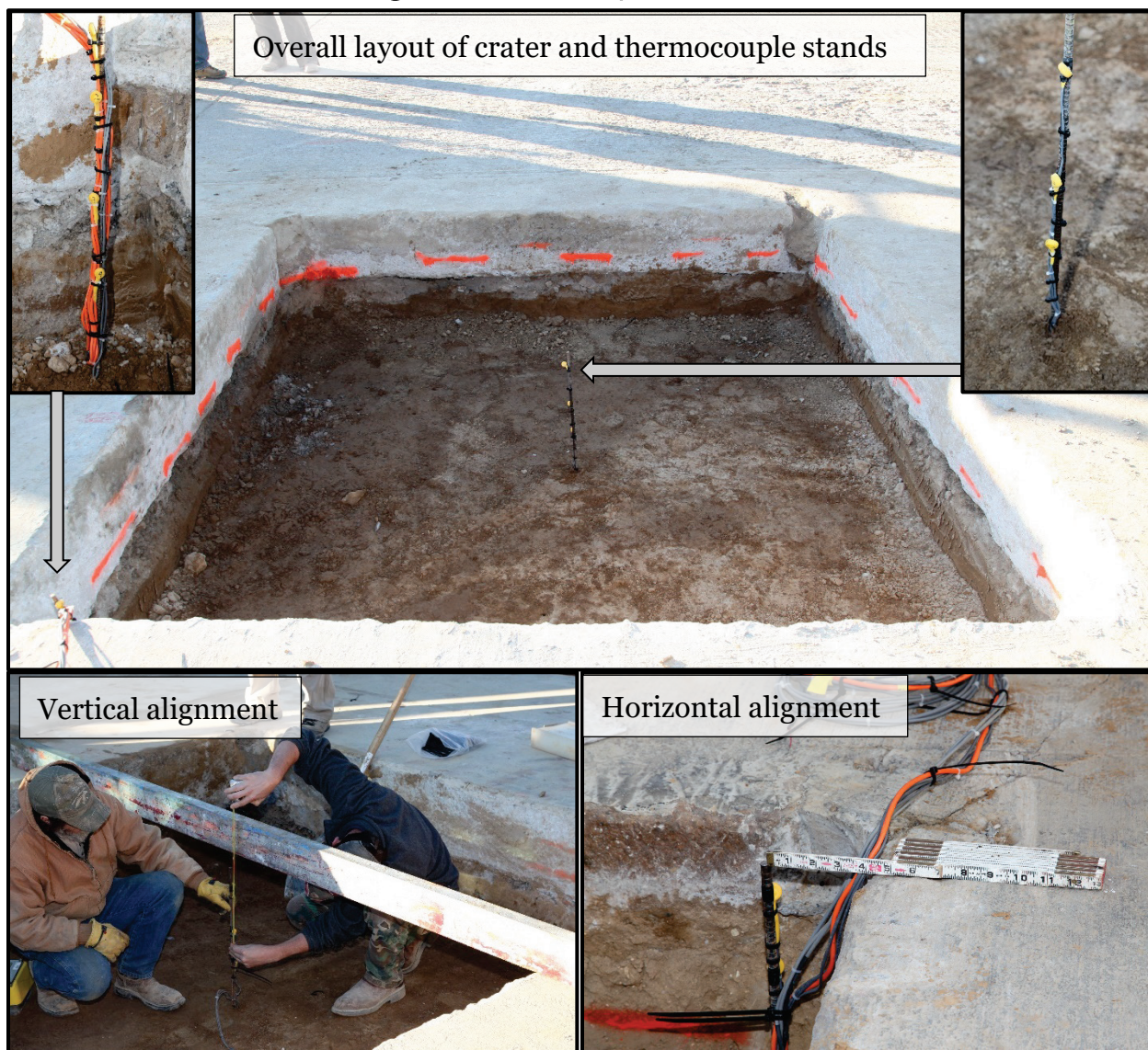


Table 9. Thermocouple placement locations.

Material Layer	Horizontal Position in Crater	Depth within Layer
Flowable Fill	Center of crater	Mid-depth
		2 in. below upper surface
	Edge, in corner 4 in. away from north and east edges	Mid-depth
		2 in. below upper surface
RS Concrete	Center of crater	Mid-depth
	Edge, in corner 4 in. away from north and east edges	Mid-depth
		2 in. below upper surface

5.1.4 Flowable fill placement

In each crater, an approximately 14-in.-deep layer of flowable fill was placed by using a volumetric mixer; the overall process is shown in Figure 15. Five supersacks of flowable fill material, as well as the appropriate mixing water (Figure 13b), were loaded into the volumetric mixer (Figure 15a). Material was placed directly into the repair crater (Figure 15b). A sample of material was obtained for testing (Figure 15c). For all six flowable fill placements, a consistent set of volumetric mixer settings was used. The water pump setting was “Flowable Fill.” For the first crater repair (Control), the material gate setting adjusted to achieve an appropriate consistency. The same material gate setting was used for all subsequent repair crater placements.

Once material was added to the target depth (Figure 15d), setting behavior of the flowable fill was assessed by periodic DCP testing at 15-min intervals after placement (Figure 15e). DCP measurements were made about 1 ft in from the crater’s south edge, and repeated DCP measurements were spaced about 1 ft apart. Once the flowable fill had strengthened to the point of DCP refusal, a surface elevation survey was conducted before the concrete cap was placed (Figure 15f).

Figure 15. CSA flowable fill placement process.

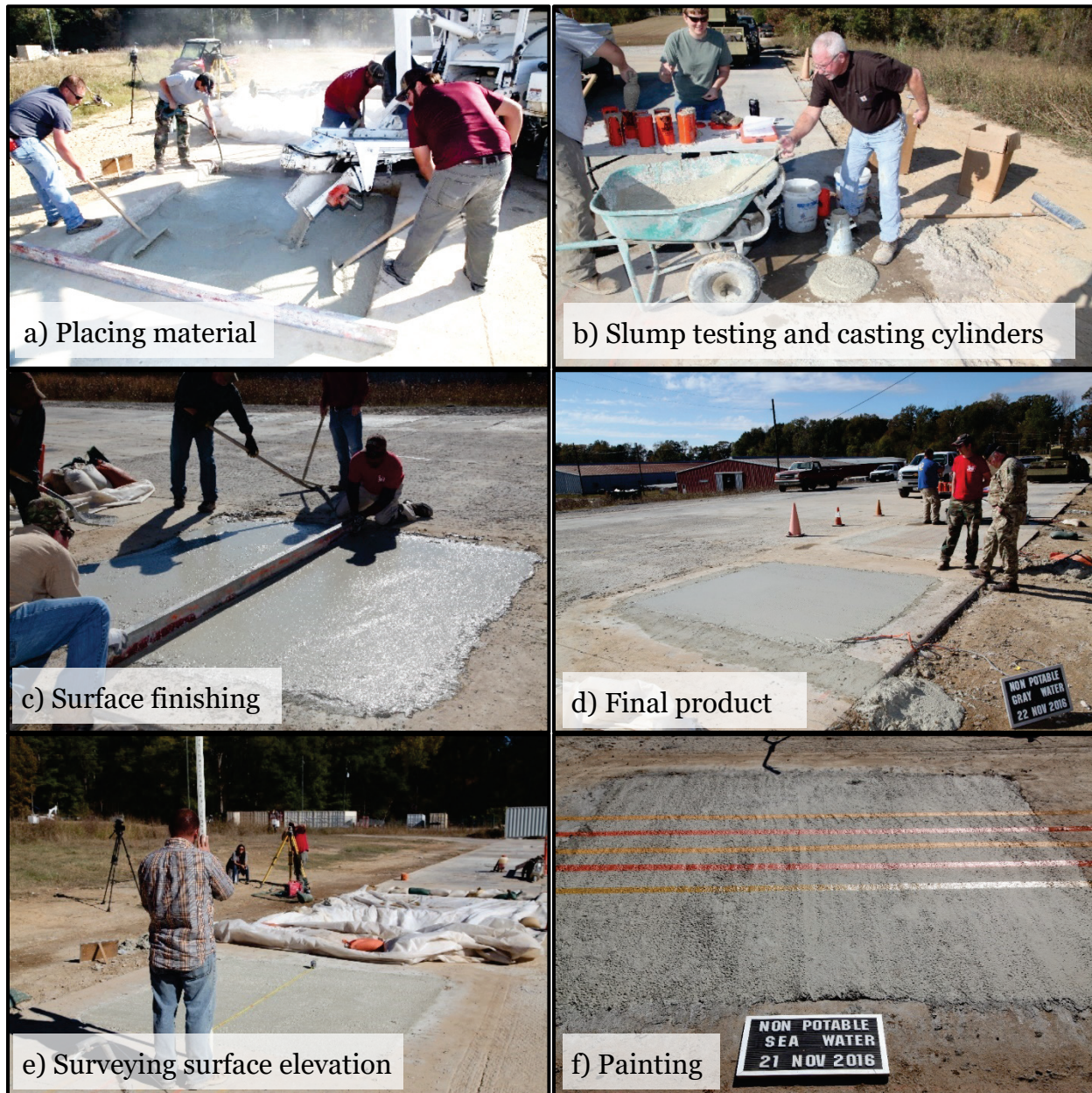


5.1.5 Concrete placement

In each crater, an approximately 10-in.-thick rapid-setting concrete cap was placed using a volumetric mixer; the overall process is shown in Figure 16. Four supersacks of flowable fill material, as well as the appropriate mixing water, were loaded into the volumetric mixer in the same manner as before (Figure 13b and 15a). Material was placed directly into the repair crater (Figure 16a). For all six concrete placements, volumetric mixer settings for the water pump were “Rapid Set.” For the first crater repair (Control), the material gate setting adjusted to achieve an appropriate consistency. The same material gate setting was used for all subsequent repair crater placements.

A sample of material was obtained and used for fresh property testing and casting cylinders for unconfined compressive strength (Figure 16b). The repair surface was screeded smooth (Figure 16c), at the conclusion of which a timer was started. The completed repair crater (Figure 16d) was allowed to cure for 2 hr. During this time, a surface elevation survey was performed (Figure 16e), and marking lines were painted on the surface to guide load cart trafficking (Figure 16f).

Figure 16. CSA concrete placement process.



5.1.6 Material testing

Samples of the flowable fill and rapid-setting concrete materials were obtained at about the midway point of each field placement. Fresh properties were measured (slump and mix temperature). From each material combination (12 sets of material and water source), nine cylindrical specimens (4 in. by 8 in.) were cast for compressive strength testing in the laboratory in accordance with ASTM C39/39-M (2016b). Specimens were left in the field until set before transport to the laboratory and cured at

standard laboratory temperature prior to testing. Three specimens each were tested at 2 hr, 24 hr, and 168 hr (7 days) after casting.

Additionally, about five months after placement, three cores (4-in. diam) were obtained from each of the rapid-setting concrete materials. These were cut down to make 4-in. by 8-in. cylinders that were tested for compressive strength in accordance with ASTM C39/39-M (2016b).

5.1.7 Trafficking

A simulated aircraft load cart was used with an aircraft tire inflated to 325 psi (2.24 MPa) and a wheel load of 34,500 lb (153.5 kN), as shown in Figure 17. Traffic was applied in a normally distributed wander pattern consisting of 16 passes across a width of 45 in. (1.14 m). Each traffic pass was 9 in. (0.23 m) away from the adjacent pass. This resulted in a pass/coverage ratio of 4.0. Additional details of the trafficking methodology are available in Priddy et al. (2016a).

Two hours after placement of each rapid-setting concrete crater cap was completed, trafficking was conducted for 112 passes (7 standard patterns) to verify the early-age performance of the materials. At the conclusion, a visual inspection was made to identify any chipping, spalling, or cracking distresses. Further, a surface elevation profile was obtained to identify any changes in surface roughness under traffic.

After all crater repairs were completed (six days after the final crater repair was placed), additional trafficking was applied for a total of 3,584 passes (224 standard patterns) on all of the repair craters. Periodically, a visual inspection was performed to look for cracks, spalling, or other surface defects. Intermediate visual inspections were conducted at cumulative pass levels of 1,024; 1,536; 2,048; 2,560; and 3,072. In keeping with the criteria used by Priddy et al. (2016a), “failure for this project was quantitatively defined by high-severity shattered slab or spalling greater than 2-ft long [sic], 6-in. wide [sic], and 2-in. deep [sic] across 50 percent of the spall length.” At the conclusion of trafficking, a final surface elevation profile was obtained. All trafficking was completed within one month after placement of the first crater repair.

Figure 17. Aircraft load cart trafficking of repair craters.



5.2 Results

5.2.1 Construction observations

During placement of the flowable fill and concrete materials, no problems were encountered using the various water sources with the volumetric mixer equipment. For the soapy water (synthetic greywater), no foaming issues were observed with the water pump or lines. However, only moderate quantities of each water source (about 500 gal) were used through the equipment. Further, each water source was loaded into the mixer's onboard storage tank in the morning; at the end of the day, any leftover water was drained out, and the water tank was flushed with clean water in preparation for the next day. Therefore, any potential equipment issues associated with long-term use of these impure water sources were not encompassed by this demonstration project. Possible concerns could include corrosion of metal parts from seawater or brackish salt water; also, settlement of any sediment in river or pond surface water sources could plug up water lines and pumps. An increased level of equipment maintenance could be required to address these issues.

The surface elevation profile surveys of subgrade surface, flowable fill surface, and concrete cap surface prior to traffic were used to determine as-constructed layer thicknesses of the CSA repair materials used in each crater. These data are presented in Table 10. For the flowable fill, target

thickness was 14 in., and actual thicknesses ranged from 13.5 to 15.1 in. with Crater 1: Control being the thickest. For the concrete capping material, target thickness was 10 in., and actual thicknesses ranged from 8.9 to 10.6 in. with Crater 1: Control being the thinnest.

Layer thickness data were combined with crater horizontal dimensions to estimate the volume of each material that was placed; this is also presented in Table 10. For flowable fill, about 3 yd³ was placed for each crater repair. For CSA concrete, about 2 yd³ was placed for each crater repair.

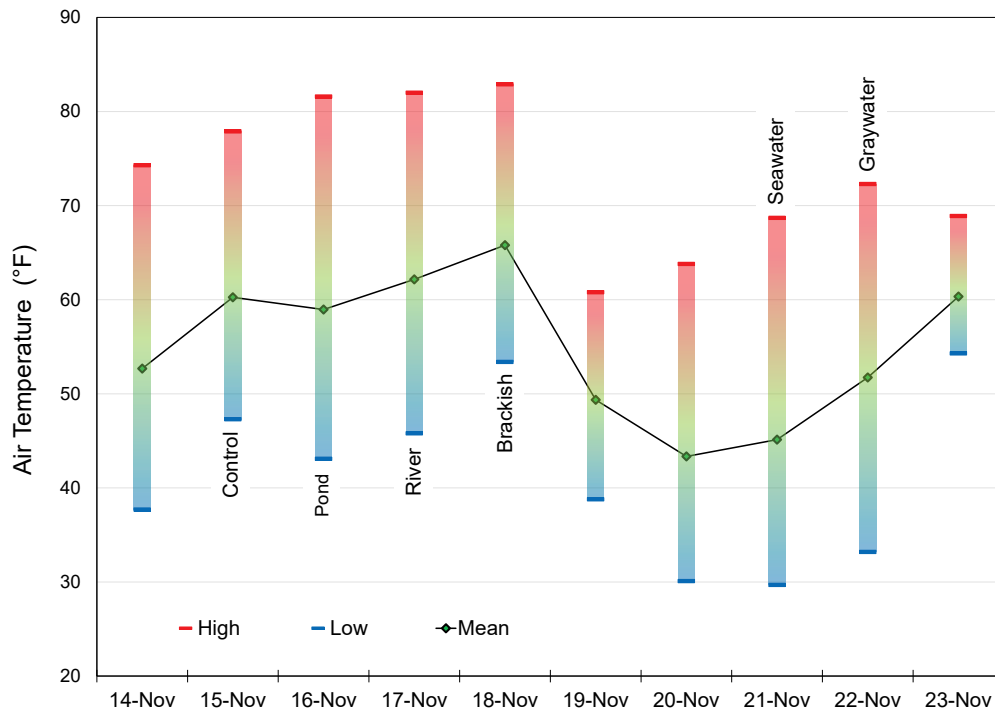
Table 10. Material placement quantities and as-constructed layer thicknesses.

Crater No.	Flowable Fill		RS Concrete	
	Volume (yd ³)	Thickness (in.)	Volume (yd ³)	Thickness (in.)
1: Control	3.2	15.1	1.9	8.9
2: Pond	2.8	13.6	2.2	10.5
3: River	2.9	14.3	2.0	9.7
4: Brackish	3.1	14.1	2.2	9.9
5: Seawater	2.8	13.5	2.2	10.5
6: Syn. greywater	2.9	13.6	2.2	10.6

5.2.2 Weather conditions

The field demonstration occurred during late fall and was spread across two work weeks. During the intermediate weekend, a cold front moved through the area, resulting in much cooler ambient temperature conditions on the final two days of the demonstration. Figure 18 presents the temperature conditions during the demonstration. For the first four repair craters (Control, Pond, River, and Brackish), the mean daily temperature was about 60 to 66°F. However, for the final two repair craters (Seawater and Synthetic Greywater), the mean daily temperature was 45 to 52°F. This did not impact construction aspects of the demonstration, but this temperature change did impact the mixtures' material properties as described in the next sections.

Figure 18. Ambient temperature conditions during field demonstration.



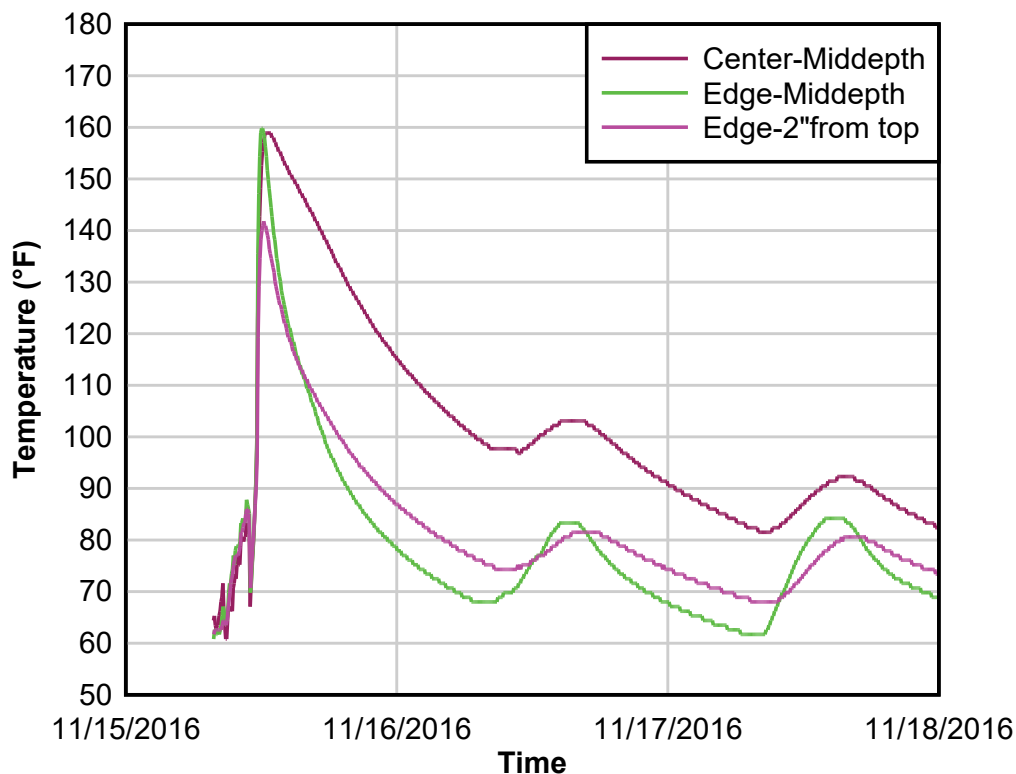
5.2.3 Temperature monitoring

An example of the temperature measurement data for the rapid-setting surfaces is shown in Figure 19. Similar trends are seen in all other craters' repairs; thus, plots are not shown for brevity. On November 15 from the time when thermocouples began recording data, the plot shows an increase in temperature as ambient air temperatures rose. Once CSA concrete was placed, the temperature dropped to about 70°F, which was the initial concrete temperature. Next, the temperature almost immediately began a rapid increase as the hydration reaction initiated and accelerated. The maximum temperature measured was about 160°F for the layer mid-depth locations in both the center and the edge of the crater repair.

For the 2 in. below surface edge of crater location, the maximum rapid-setting concrete temperature was lower, about 140°F. This is likely due to lower thermal mass near the edge and loss of heat to the surrounding pavement. The maturity calculation results of Carruth et al. (2019) suggest that soon after placement, the near-surface edge locations of the crater repair will have lower compressive strength than the mid-depth of the concrete mass.

Once the initial peak rapid-setting concrete temperature occurred, temperature began to decrease. This occurred more rapidly for the edge of the crater locations than for the center of the crater repairs; once again, this was likely due to the lower thermal mass near the edge and the loss of heat to the surrounding pavement. Over the course of about five to seven days, temperature of the rapid-setting concrete reached ambient conditions as the hydration reaction slowed and removed the internal heat source.

Figure 19. CSA concrete cap temperature history by location for Crater 1: Control.



Of particular interest to this study is the magnitude of the initial temperature spike of the rapid-setting concrete material and how long this takes to occur. A summary of the relevant parameters is provided in Table 11 for CSA concrete. Note the initial temperature of the control was 69°F (as seen in Figure 19), and for the repairs mixed using pond, river, and brackish waters, the initial temperatures were within about 3°F of the control. However, for the repairs mixed using seawater and greywater, the initial temperature was more than 10°F lower than the control. This is most likely due to the lower ambient temperature conditions just prior to and during the placement of those materials (Figure 18). The water samples and CSA dry materials were all located in unheated storage areas during this time, so their temperatures would be relatively low.

Interestingly, the maximum rapid-setting concrete temperatures for surfaces mixed using seawater and greywater were very similar to the control. However, the time required for them to achieve peak temperature was about 50% longer than for the control; this is also seen in the lower slope of the temperature peak. In general, this suggests that the lower initial material temperature resulting in lower initial concrete temperature resulted in a slower rate of hydration initially, but ultimately the hydration reaction caught up.

Table 11. CSA concrete temperature summary.

Crater No.	Initial Temperature ^a (°F)	Maximum Temperature ^b (°F)	Elapsed Time (min)	Slope to Peak (°F/hr)
1: Control	69.2	158.9	80	68.9
2: Pond	65.9	166.1	90	67.2
3: River	66.2	164.3	85	69.9
4: Brackish	70.1	168.8	95	62.5
5: Seawater	57.8	158.9	125	48.4
6: Syn. Greywater	59.0	159.8	120	51.8

^a Average of all thermocouples.

^b Measured at center of crater, layer mid-depth location.

A summary of temperature observations for CSA flowable fill is provided in Table 12. Note the lower initial temperatures for the repairs mixed with seawater and greywater compared to that of the control and also the lower slope of the temperature peak. As discussed above, this is likely due to low ambient temperature conditions.

Table 12. CSA flowable fill temperature summary.

Crater No.	Initial Temperature ^a (°F)	Maximum Temperature ^b (°F)	Elapsed Time (min)	Slope to Peak (°F/hr)
1: Control	62.8	114.8	150	20.5
2: Pond	54.0	106.7	155	21.3
3: River	58.8	107.6	110	25.5
4: Brackish	62.5	115.7	115	27.7
5: Seawater	49.8	94.1	140	18.5
6: Syn. Greywater	51.6	97.7	145	19.7

^a Average of all thermocouples.

^b Measured at center of crater, layer mid-depth location.

5.2.4 Material properties testing

5.2.4.1 Fresh properties and set time

Fresh properties of temperature and slump were measured from each sample of material obtained during placement, and the data are provided in Table 13. Unfortunately, some of the data were lost; however, the most important item to note is the difference in temperature between the Crater 1: Control material and Craters 5 and 6. Temperature for Crater 5: Seawater was almost 20°F lower than for the control. Similarly, temperature for Crater 6: Synthetic Greywater was almost 15°F lower than for the control. These observations confirm the trends in initial material temperature described in the previous section.

Table 13. Fresh properties: Mixture temperature and slump.

Crater No.	Flowable Fill		RS Concrete	
	Mix Temp (°F)	Slump (in.)	Mix Temp (°F)	Slump (in.)
1: Control	67.9	12	74.0	11
2: Pond	60.9	---a	---a	---a
3: River	---a	---a	---a	---a
4: Brackish	---a	---a	---a	---a
5: Seawater	49.4	12	54.4	9.25
6: Syn. Greywater	54.1	12	59.2	11

(a) Data missing.

A summary of the DCP measurements made in CSA flowable fill is presented in Table 14. For the materials tested at higher ambient temperatures (control, pond, river, and brackish water), there is generally a sharp increase in CBR at 30 or 45 min after placement. This falls within the range of initial and final set times observed in laboratory testing (Section 4.2.3). For seawater and greywater crater repairs placed at lower temperatures, a jump in CBR is not observed until 60 to 90 min after placement. This delayed strength gain behavior was not observed in laboratory testing performed at controlled temperature conditions (Section 4.2.3), which suggests that the lower temperature is the most likely cause.

Table 14. Flowable fill strength gain measurement by DCP test.

Crater No.	Depth (in.)	CBR (%) at time after placement (min)						
		15	30	45	60	75	90	105
1: Control	0-6	8	30	35	55	55	---a	---a
	6-12	22	75	80	80	90	---a	---a
2: Pond	0-6	0	1	50	50	50	---a	---a
	6-12	4	40	60	80	100	---a	---a
3: River	0-6	2	22	40	DCP equipment broken		---a	---a
	6-12	7	35	85			---a	---a
4: Brackish	0-6	2	21	50	60	50	---a	---a
	6-12	5	50	100	100	100	---a	---a
5: Seawater	0-6	0	0	5	30	30	50	50
	6-12	0	0	10	30	50	80	100
6: Syn. Greywater	0-6	0	5	18	30	50	50	75
	6-12	2	8	22	50	60	75	100

(a) Not tested.

For the CSA concrete, visual observations of set time were made every 15 min and are presented in Table 15. Generally, the material was set by 30 to 60 min after placement for all water sources. This agrees with the laboratory results (Section 4.2.1).

Table 15. Visual observations on set time for CSA concrete.

Crater No.	Firm to Touch, Set Complete	Color Change Complete
1: Control	30 min	45 min
2: Pond	60 min	75 min
3: River	45 min	60 min
4: Brackish	30 min	45 min
5: Seawater	60 min	75 min
6: Syn. Greywater	45 min	> 60 min and < 105 min

5.2.4.2 CSA flowable fill hardened properties

Results of compressive strength testing of CSA flowable fill are located in Table A6 of the appendix, and a summary is presented in Figure 20. In

general, strength increases over time is as expected. To rank the effects of water source on compressive strength at each time, one-way ANOVAs were conducted, and the Dunnett multiple comparison tool was used to evaluate whether the results were statistically significantly different from the control at the 95% confidence level (Dunnett 1964); this is summarized in Table 16. At 2-hr age, only seawater had a significantly lower strength than the control. Since seawater was not significantly lower in controlled temperature laboratory testing (Section 4.2.4), lower initial temperature in the field demonstration likely contributed to this result. Further, at 24 and 168 hr after placement, all water sources were significantly stronger than the control, which also suggests that an initial delay of hydration due to lower temperature was subsequently recovered. All specimens met the minimum 750-psi compressive strength requirement after 24 hr of curing.

Figure 20. Summary of CSA flowable fill compressive strengths.

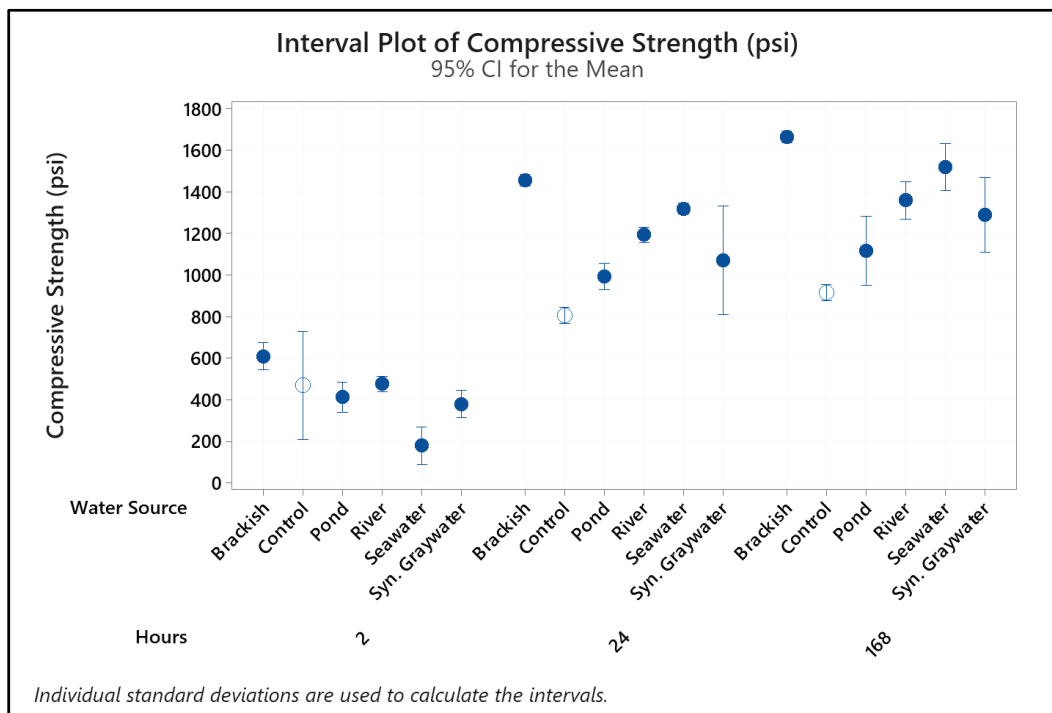


Table 16. Dunnett multiple comparison procedure results for CSA flowable fill.

Water Source	N	2 hr		24 hr		168 hr	
		Mean	Group	Mean	Group	Mean	Group
1: Control	3	470	A	807	A	916	A
2: Pond	3	413	A	993		1,117	
3: River	3	477	A	1,193		1,360	
4: Brackish	3	610		1,457		1,663	
5: Seawater	3	180		1,320		1,520	
6: Syn. Greywater	3	380	A	1,070		1,290	

Means not labeled with the letter A are significantly different from the control with 95% confidence.

5.2.4.3 CSA concrete hardened properties

Results of compressive strength testing of CSA flowable fill are located in Tables A6 and A7 of the appendix, and a summary is presented in Figure 21. In general, strength increases over time as expected. To rank the effects of water source on compressive strength at each time, one-way ANOVAs were conducted, and the Dunnett multiple comparison tool was used to evaluate whether the results were statistically different from the control (Dunnett 1964); this is summarized in Table 17. At 2-hr age, brackish, seawater, and greywater are significantly lower than the control; of those, seawater is noticeably lower than the other two. However, at 24 hr, all three are not statistically different from the control. At 24-hr age, only river water is lower than the control. At 168 hr, only brackish water is lower than the control. For the approximately five-month-age strength measurements, no water sources were significantly different from the control. Since carefully controlled laboratory testing (Section 4.2.2) did not find any statistically significant differences from the control at 2, 24, or 168 hr, it is reasonable to conclude that strength differences in the field demonstration are most likely due to temperature differences and material or sampling variability.

Figure 21. Summary of CSA concrete compressive strengths.

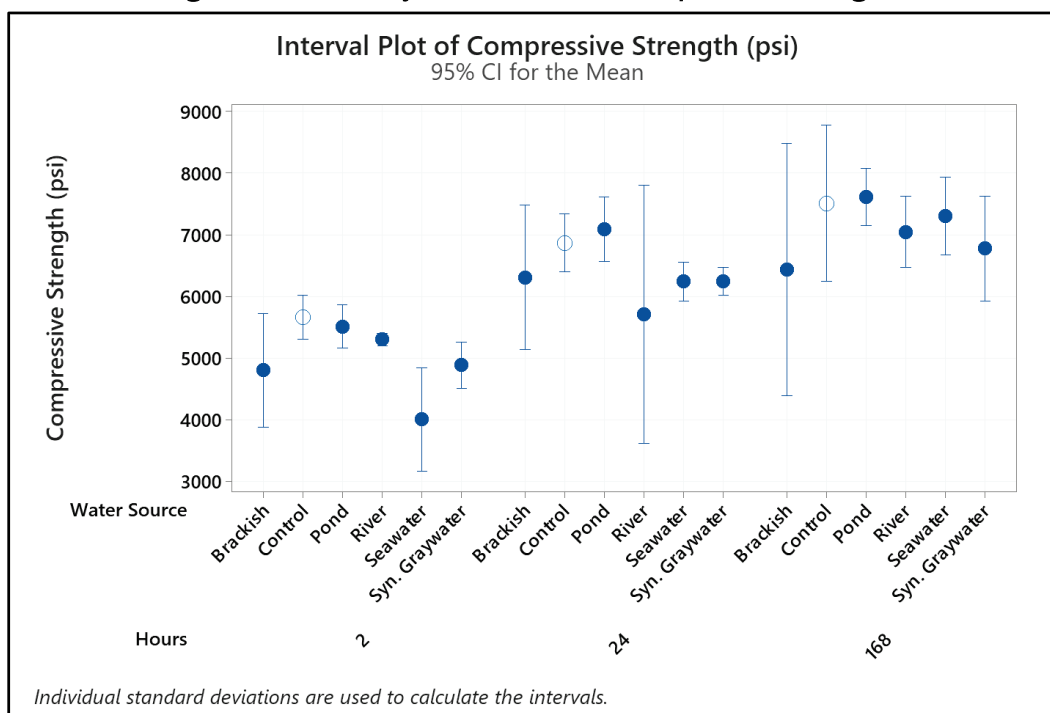


Table 17. Dunnett multiple comparison procedure results for CSA concrete.

Water Source	N	2 hr		24 hr		168 hr		153-160 days	
		Mean	Group	Mean	Group	Mean	Group	Mean	Group
1: Control	3	5,663	A	6,867	A	7,510	A	11,040	A
2: Pond	3	5,513	A	7,090	A	7,610	A	10,367	A
3: River	3	5,303	A	5,707		7,043	A	11,020	A
4: Brackish	3	4,803		6,307	A	6,440		10,663	A
5: Seawater	3	4,007		6,243	A	7,303	A	10,050	A
6: Syn. Greywater	3	4,890		6,247	A	6,780	A	9,773	A

Means not labeled with the letter A are significantly different from the control with 95% confidence.

5.2.5 Trafficking results

Visual inspection of the crater repairs after initial trafficking for 112 passes conducted 2 hr after placement revealed no surface distresses or damage at the edges. This indicates that any initial increased set times or retarded strength gains of the materials were not of large enough magnitude to inhibit a 2-hr-return-to-service requirement for repair craters.

Additional trafficking was conducted on all repair craters. Results of the periodic visual inspections are located in Tables A9 to A14 of the appendix. At 1,024 passes, there were spalls or edge cracks on all sides of all repair craters. No spalls exceeded a 1-in. depth or 3.5-in. width, and none of the repairs exceeded the failure criteria. At 1,536 passes, no spalls exceeded a 1.5-in. depth or 6-in. width, and none of the repairs were classified as failed. Both the control and seawater crater repairs had low-severity cracks extending full width across the repairs. At 2,048 passes, no spalls exceeded a 1.75-in. depth or 8-in. width, and none of the repairs were classified as failed. The same results were observed at 2,560 passes. At 3,072 passes, no spalls exceeded 2.25 in. deep or 10 in. wide, and none of the repairs were failed.

At the conclusion of trafficking (3,584 passes), none of the repair craters met or exceeded the failure criteria. Throughout all of the trafficking, the main distress observed was spalling at edges of the repairs. Figure 22 through 27 present photographs of the edges of each repair crater. Generally, the largest amount of spalling occurred on the west and east edges of the repairs. This makes sense because load cart traffic was applied in an east-west direction, so these edges were directly impacted by the aircraft tire. This observation is consistent with typical performance behavior of repair crater traffic tests (Priddy et al. 2016a). The other edges also exhibited cracking and spalling. Also, close observation of the load cart during operation showed that the repairs had some rocking movement during loading and unloading. The lack of load transfer between the repair and the surrounding pavement is the primary reason for these observations and aligns with those reported in Priddy et al. (2016a).

A final surface elevation survey was conducted after trafficking was complete. The differential movements compared to the initial survey before trafficking were calculated. Table 18 summarizes these data. The maximum surface changes were about 1 in. or less, which indicates low likelihood of surface irregularities' impacting aircraft operations. Overall, the trafficking data suggest that all of the crater repairs made with impure mixing water performed comparably to the control.

Table 18. Surface roughness summary at conclusion of trafficking.

Crater No.	Maximum Elevation Differential (in.)	Location of Maximum Differential
1: Control	0.86	East and West Edges
2: Pond	0.20	All Edges
3: River	0.68	East Edge
4: Brackish	0.68	West Edge
5: Seawater	0.44	West Edge
6: Syn. Greywater	1.04	West Edge

Figure 22. Condition of repair edges after trafficking for Crater 1: Control.

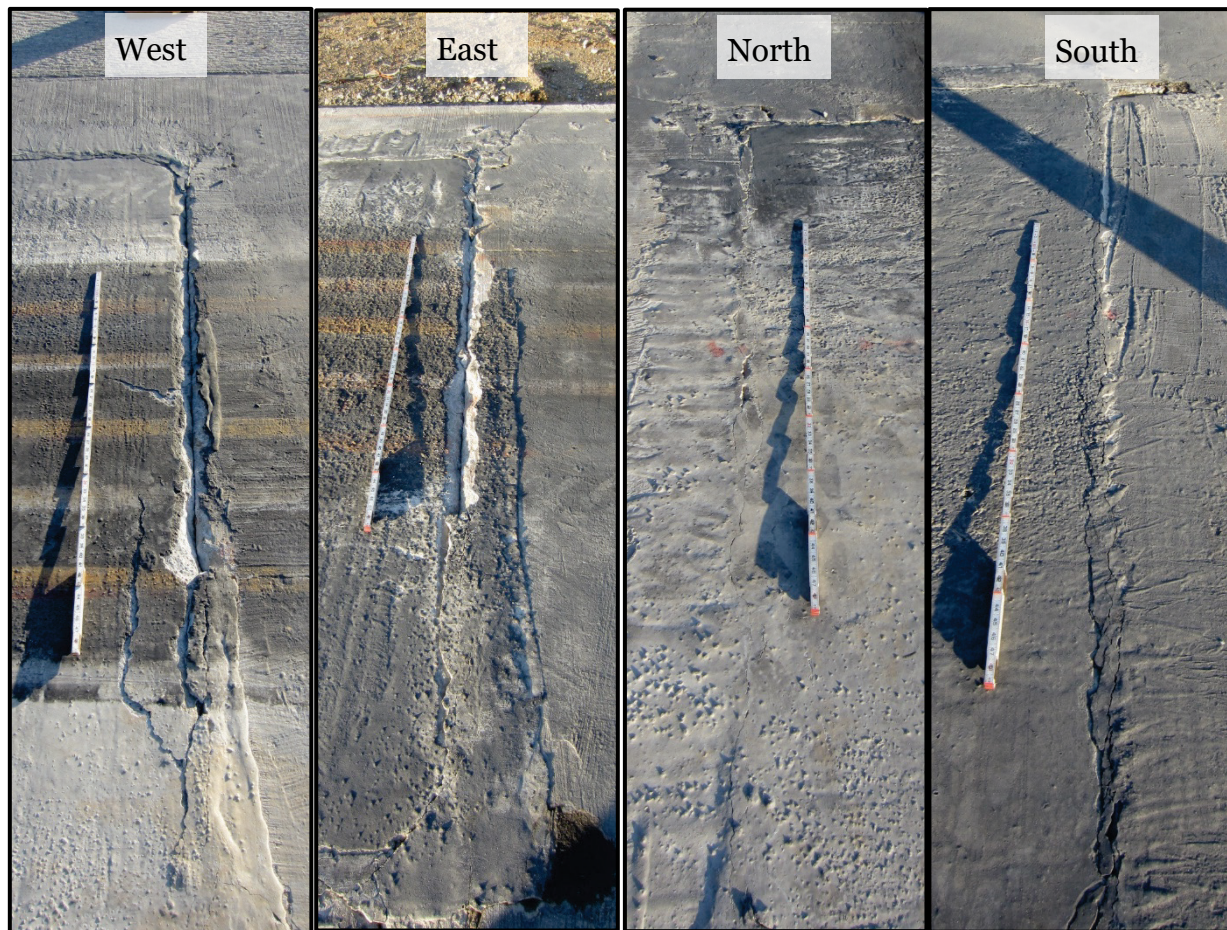


Figure 23. Condition of repair edges after trafficking for Crater 2: Pond.

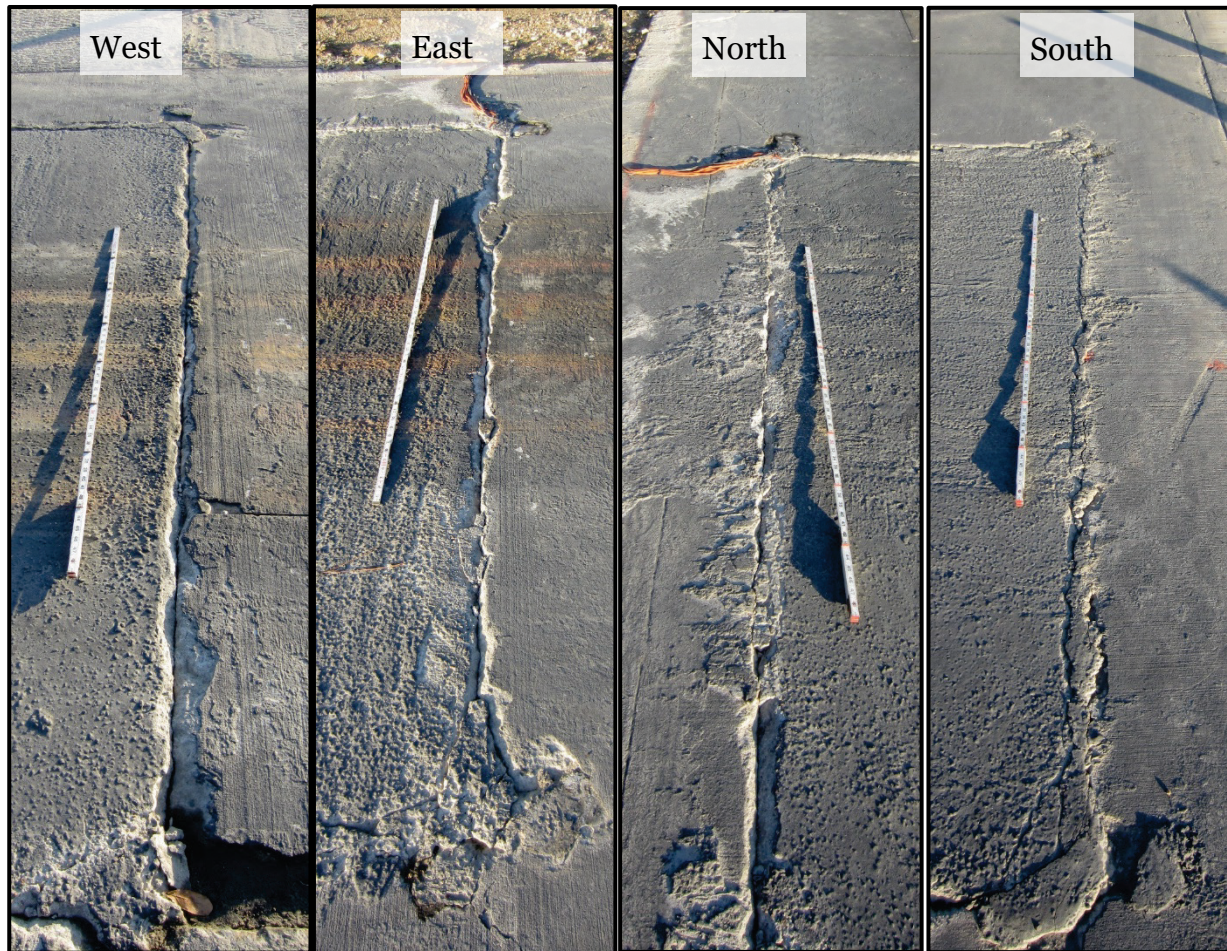


Figure 24. Condition of repair edges after trafficking for Crater 3: River.

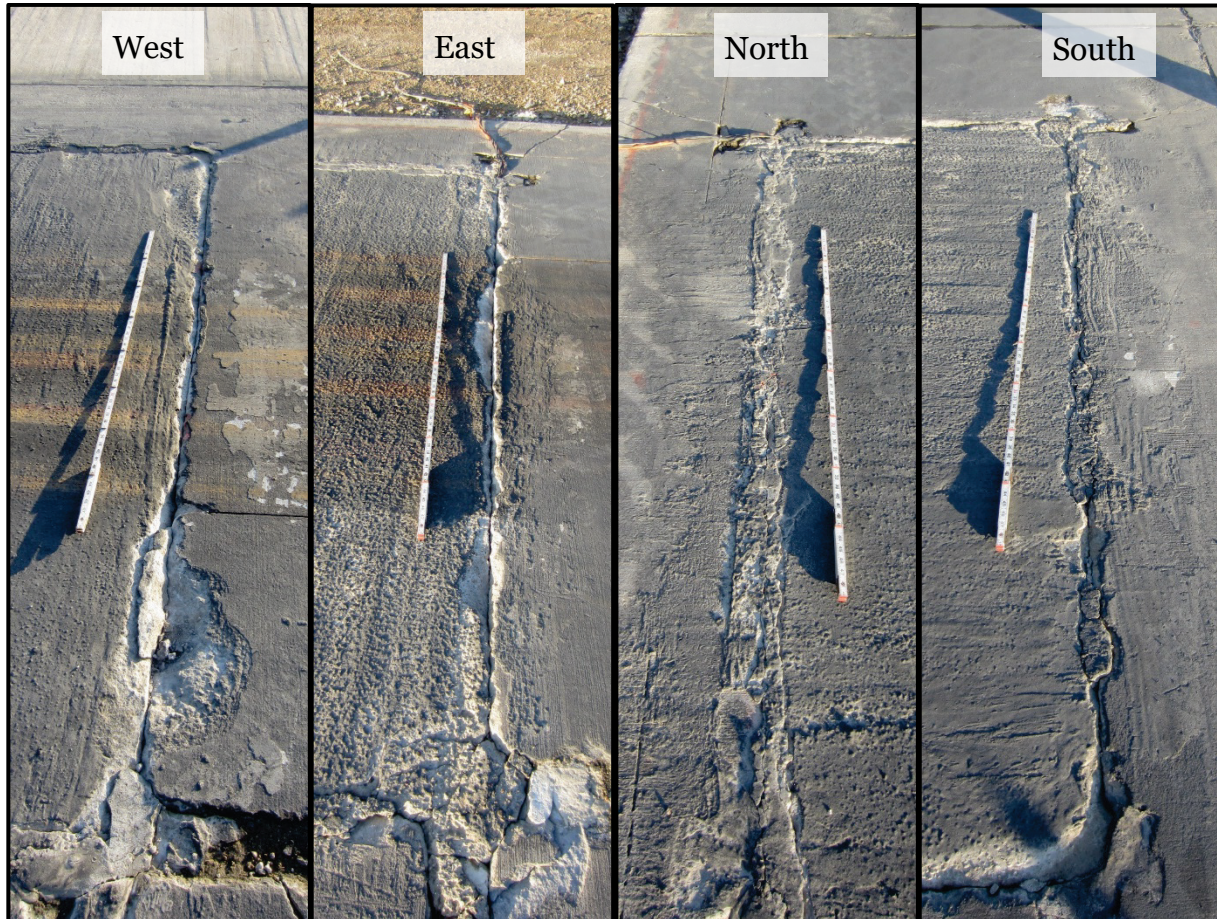


Figure 25. Condition of repair edges after trafficking for Crater 4: Brackish.

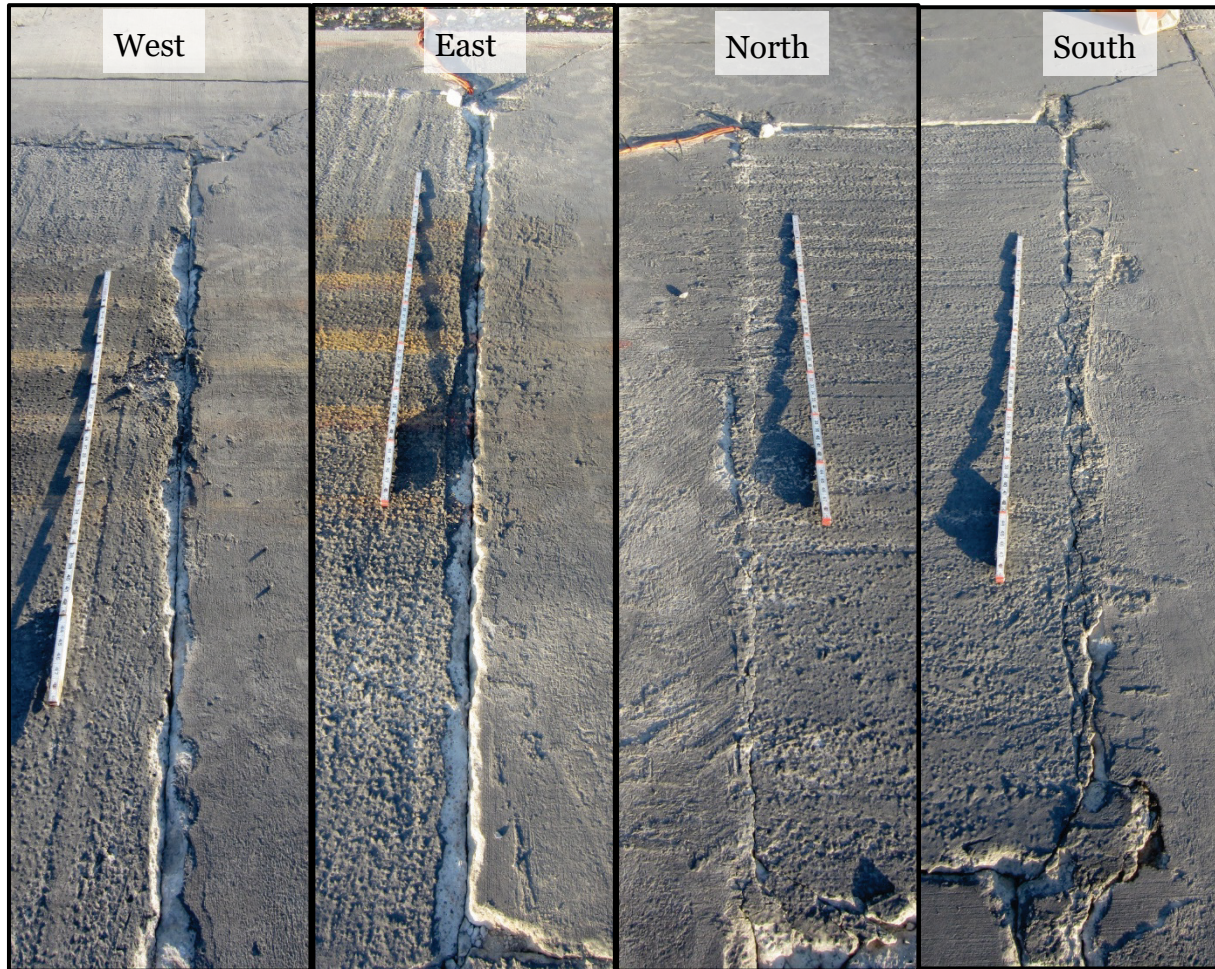


Figure 26. Condition of repair edges after trafficking for Crater 5: Seawater.

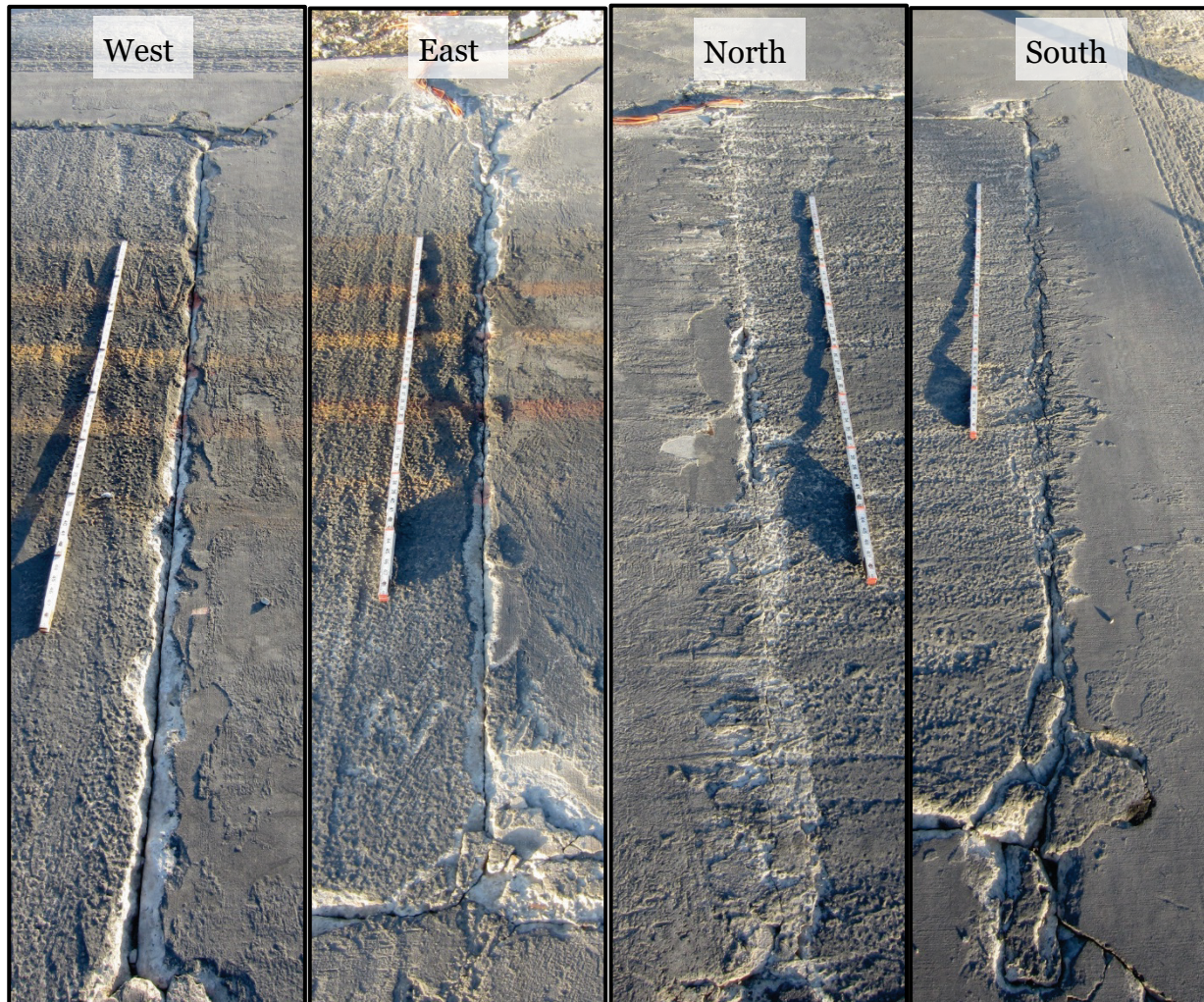
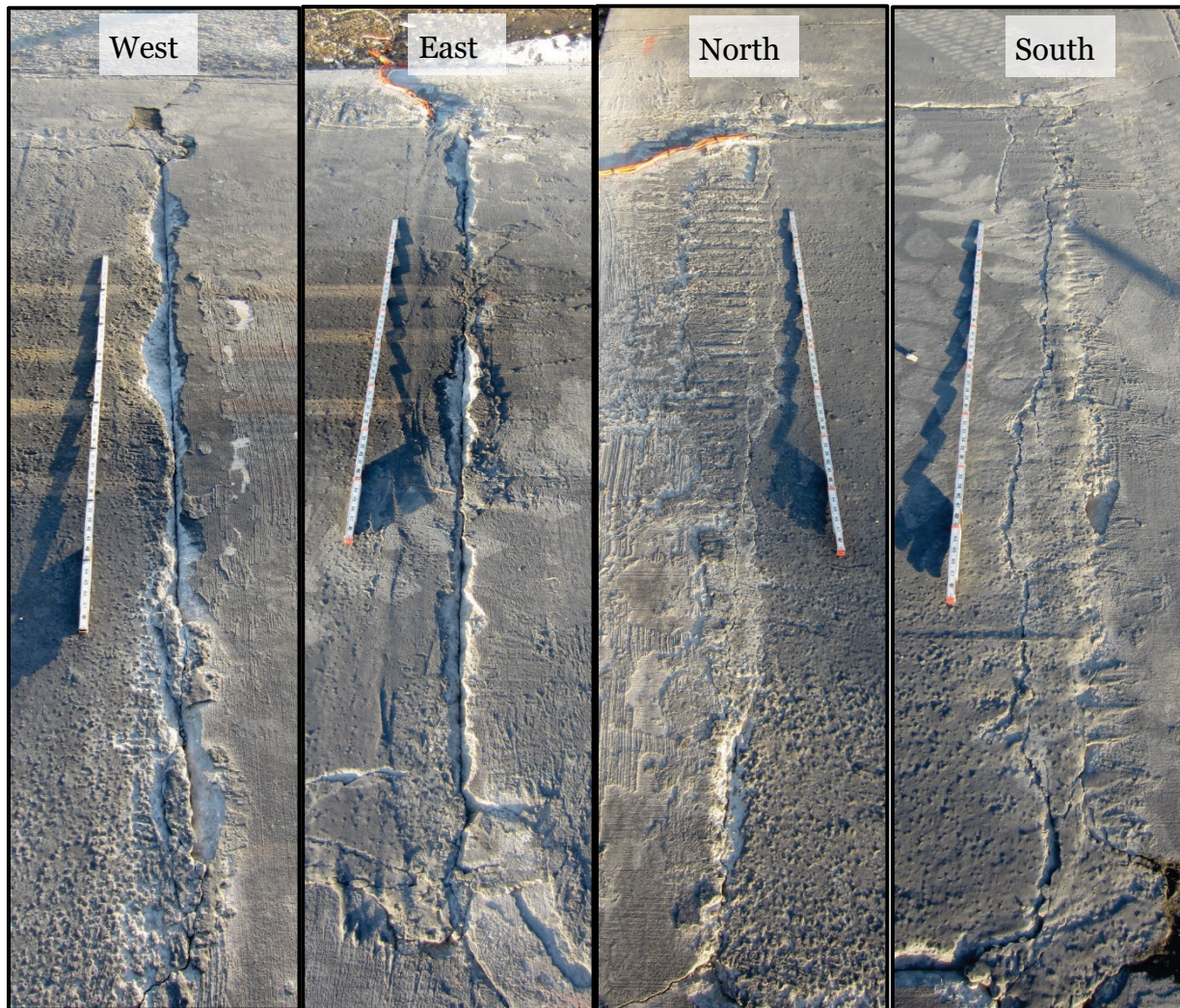


Figure 27. Condition of repair edges after trafficking for Crater 6: Syn. greywater.



6 Discussion and Recommendations

6.1 Laboratory study summary

This laboratory research effort studied the effects of impure water sources on the early-age strength development of commercially available CSA cement-based concrete products. Twelve synthetic water solutions were prepared to isolate the effects of specific ions commonly found in impure water sources on the early-age strength development of a CSA cement-based concrete product.

Of these water solutions, those prepared using calcium chloride had the greatest retarding effect on strength development. However, for the retarding effect to be notable enough to impact construction, a relatively concentrated source of calcium chloride was required. Concentrated solutions of calcium chloride do occur in nature, such as in the Dead Sea, the Great Salt Lake, briny wells, and small pools of water contaminated by de-icing salts. Outside of these specific instances, it is uncommon for water to contain such high concentrations of calcium chloride. Additionally, even the concrete that was prepared using high concentrations of calcium chloride in the mixing water obtained unconfined compressive strength equal to or greater than that of concrete prepared using the control at just 168 hr after mixing.

This same retarding effect was not noticeable in synthetic solutions prepared using sodium chloride, sodium sulfate, calcium sulfate, or synthetic seawater. Unconfined compressive strengths at 2, 24, and 168 hr each appeared to be elevated by solutions containing potassium chloride, magnesium chloride, and magnesium sulfate.

None of the natural water or greywater sources in this study substantially altered the strength development of the CSA-based concrete material, and this observation was supported by the synthetic solutions included in this study.

It was difficult to determine the time of set of the CSA cement-based flowable fill material, and these results varied greatly. This was likely due in part to the inherently low strength of the material and variations in the cementitious content of the prepackaged product. However, it did appear that both the seawater and greywater solutions accelerated

the setting behavior and early-age strength development of this material. Additional testing of the cementitious material used to produce the flowable fill material is recommended, using a higher-strength product, as the strength development of this product may be more sensitive to the use of impure sources of mixing water.

6.2 Application to RADR

The field demonstration found no significant problems associated with the use of impure mixing water for RADR. For two of the repair craters, strength gain was slowed down compared to the control. However, the evidence suggests that the cause was the lower ambient temperature conditions on those days, not the mixing water. Load cart traffic testing indicated no differences in structural or functional performance of the repair craters made with impure mixing water compared to the control.

Although not explicitly examined in this research effort, the use of brine reject water from reverse osmosis water purification units (ROWPU) is also of interest for RADR. This issue was previously studied by Lovingood et al. (2015), who measured two separate samples of ROWPU brine and found that the concentrations of sodium, potassium, magnesium, and calcium cations, as well as chlorine and sulphate anions, were all lower than those for seawater. Their study further found that use of ROWPU brine as mixing water did not negatively impact the performance of CSA flowable fill or CSA concrete at early ages relative to potable water. Since ROWPU brine has been measured to have lower concentrations of the important cations and anions than seawater, for this study it was assumed that seawater would be a worst-case scenario relative to ROWPU brine. In other words, if seawater provides acceptable performance, there are no data to suggest that ROWPU brine would not be acceptable also.

6.3 Recommendations for RADR

Based on the findings of this study, the following recommendations are made for implementing use of impure water sources as mixing water with CSA-cement repair materials for RADR.

- The use of most commonly available impure water sources can safely be allowed for RADR with the standard CSA cement products (flowable fill and Rapid Set concrete) tested herein. This can include seawater,

- ROWPU brine reject water, brackish water, soapy greywater, collected rainwater, and naturally occurring surface water sources.
- Water sources with high concentrations of calcium chloride can retard strength gain of CSA cement-based products.
 - Water sources with sufficiently high calcium chloride concentrations are uncommon in most parts of the world; for water sources more briny than seawater, water quality testing is recommended before use.
 - An upper limit for calcium chloride concentrations of 2,000 mg/L is recommended to ensure early-age (at 2 hr) compressive strength reduction does not exceed 10%.
 - Highly briny water could still be used in an emergency, but the 2-hr-return-to-service requirement would be negatively impacted. Strength gain would catch up within a few hours to a few days, depending on calcium chloride concentration.
 - Methods used to collect impure water sources in the field should be assessed prior to a need for them. Issues to consider include following.
 - Use of a screen or simple filter system to prevent large debris from entering the volumetric mixer water tanks;
 - Concerns for accessing the water source, such as crossing marshy ground; and
 - Possible use of disinfectant (e.g., bleach) for any water sources that might have unhealthful bacteria.
 - Longer-term use of impure water sources will likely result in maintenance issues for the volumetric mixer. An increased maintenance frequency is suggested to mitigate this.
 - For salty water sources, frequent washing of equipment with freshwater to reduce corrosion of metal parts.
 - For any impure water source, using clean water to periodically drain and flush the water supply tank, water pump, pressure washer, pressure washer supply tank, and all piping to remove buildup of sediment or salt.

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Appendix A: Data Tables

Table A1. Early-age unconfined compressive strength of CSA products mixed with natural water and greywater sources.

Test Age (hr)	Water Source	CSA-Flowable Fill				CSA-Concrete			
		Mean (MPa)	Std. Dev. (MPa)	N	Pooled Std. Dev. (MPa)	Mean (MPa)	Std. Dev. (MPa)	N	Pooled Std. Dev. (MPa)
2 h	C (DI)	2.4	0.7	11	0.98	20.4	0.6	6	2.01
	N(S)	4.0	0.3	4	**	20.4	1.9	4	
	N(MR)	2.0	1.6	4		20.9	2.7	4	
	N(P)	2.1	0.2	7		21.2	1.0	4	
	N(G)	3.4	0.5	7	**	21.2	1.4	4	
	N(RW)	2.9	0.8	10		21.1	2.7	8	
	N(B)	2.4	0.1	10		22.5	2.1	6	
24 h	C (DI)	4.7	0.5	9	0.87	27.3	2.1	8	2.90
	N(S)	6.1	0.5	4	**	27.3	0.5	4	
	N(MR)	5.4	0.9	4		28.4	3.8	6	
	N(P)	4.9	0.3	6		29.0	1.5	4	
	N(G)	7.7	1.0	6	**	26.2	3.6	4	
	N(RW)	6.2	1.2	6	**	30.5	2.8	10	
	N(B)	5.1	1.2	6		31.0	3.9	6	
168 h	C (DI)	5.0	0.8	11	0.98	30.7	2.4	11	2.87
	N(S)	8.5	1.6	4	**	32.0	1.4	8	
	N(MR)	5.7	1.2	4		31.5	2.2	4	
	N(P)	4.1	1.3	7		30.1	3.8	4	
	N(G)	6.6	1.6	7	**	31.6	2.4	4	
	N(RW)	3.6	0.3	10	*	33.0	0.9	4	
	N(B)	5.8	0.2	10		30.3	5.1	6	

* statistically different from control, lower strength

** statistically different from control, higher strength

Table A2. Time of set of CSA products mixed with natural water and greywater sources.

Time of Set (min)	Water Source	CSA-Flowable Fill				CSA-Concrete			
		Mean (Min)	Std. Dev. (Min)	N	Pooled Std. Dev. (Min)	Mean (Min)	Std. Dev. (Min)	N	Pooled Std. Dev. (Min)
Initial	C (DI)	26.7	6.4	6	7.3	47.0	3.1	5	3.6
	N(S)	19.0	6.1	3		47.0	2.0	4	
	N(MR)	27.3	9.5	3		45.3	2.1	3	
	N(P)	27.3	9.3	3		46.0	1.7	3	
	N(G)	17.0	7.0	3		46.0	3.6	3	
	N(RW)	19.0	8.6	4		44.4	2.6	5	
	N(B)	27.0	1.0	3		45.5	6.8	4	
Final	C (DI)	50.0	14.5	6	16.7	48.2	2.7	5	2.9
	N(S)	34.0	8.7	3		48.3	2.2	4	
	N(MR)	44.7	18.0	3		46.7	1.5	3	
	N(P)	52.0	14.1	3		47.3	2.1	3	
	N(G)	34.7	16.0	3		47.3	3.1	3	
	N(RW)	46.5	24.4	4		45.4	2.6	5	
	N(B)	60.3	15.9	3		46.3	4.5	4	

*statistically different from control, lower strength

**statistically different from control, higher strength

Table A3. Unconfined compressive strength of CSA cement-based concrete prepared using synthetic water solutions for mixing water at 2 hr.

Water Source	Mean (MPa)	Std. Dev. (MPa)	Coeff. of Var.	N	Pooled Std. Dev. (MPa)
C (DI)	19.9	2.0	10.1	8	1.6
S(CC0)	19.4	3.3	17.0	4	
S(CC7)	11.0	0.3	2.7	4	*
S(CC29)	7.0	0.4	5.7	4	*
S(CS7)	25.5	1.0	3.9	4	**
S(KC7)	25.9	0.5	1.9	4	**
S(MC4)	22.8	0.7	3.1	4	
S(MS4)	26.7	0.6	2.2	4	**
S(NC8)	24.8	0.5	2.0	4	**
S(NC11)	20.3	0.4	2.0	4	
S(NC50)	20.1	0.8	4.0	4	
S(NS1)	17.2	0.7	4.1	4	
S(S)	20.4	3.0	14.7	6	

*statistically different from control, lower strength

**statistically different from control, higher strength

Table A4. Unconfined compressive strength of CSA cement-based concrete prepared using synthetic water solutions for mixing water at 24 hr.

Water Source	Mean (MPa)	Std. Dev. (MPa)	Coeff. of Var.	N	Pooled Std. Dev. (MPa)
C (DI)	27.3	2.1	7.7	8	2.0
S(CC0)	31.3	1.6	5.1	4	**
S(CC7)	31.2	1.6	5.1	4	**
S(CC29)	21.8	1.2	5.5	4	*
S(CS7)	29.9	2.3	7.7	4	
S(KC7)	33.8	0.6	1.8	4	**
S(MC4)	32.3	1.8	5.6	4	**
S(MS4)	31.7	0.7	2.2	4	**
S(NC8)	23.4	4.3	18.4	4	*
S(NC11)	27.1	1.1	4.1	4	
S(NC50)	22.3	3.0	13.5	4	*
S(NS1)	26.4	1.3	4.9	4	
S(S)	30.4	1.9	6.3	6	

*statistically different from control, lower strength

**statistically different from control, higher strength

Table A5. Unconfined compressive strength of CSA cement-based concrete prepared using synthetic water solutions for mixing water at 168 hr.

Water Source	Mean (MPa)	Std. Dev. (MPa)	Coeff. of Var.	N	Pooled Std. Dev. (MPa)
C (DI)	30.7	2.4	7.8	11	1.7
S(CC0)	32.9	2.7	8.2	4	
S(CC7)	33.7	0.7	2.1	4	**
S(CC29)	38.7	0.7	1.8	4	**
S(CS7)	30.4	1.2	3.9	4	
S(KC7)	37.2	2.2	5.9	4	**
S(MC4)	35.6	0.8	2.2	4	**
S(MS4)	34.7	1.2	3.5	4	**
S(NC8)	30.5	0.5	1.6	4	
S(NC11)	30.0	1.2	4.0	4	
S(NC50)	31.0	0.7	2.3	4	
S(NS1)	30.5	1.2	3.9	4	
S(S)	30.7	1.8	5.9	6	

*statistically different from control, lower strength

**statistically different from control, higher strength

Table A6. Field cast specimens of CSA-flowable fill unconfined compressive strength results.

Crater No.	Rep	Compressive Strength (psi) by Age (hr)		
		2 hr	24 hr	168 hr
1-Control	1	400	820	930
	2	420	810	920
	3	590	790	900
2-Pond	1	430	990	1,190
	2	380	1,020	1,100
	3	430	970	1,060
3-River	1	480	1,180	1,390
	2	490	1,190	1,370
	3	460	1,210	1,320
4-Brackish	1	620	1,450	1,650
	2	580	1,470	1,670
	3	630	1,450	1,670
5-Seawater	1	220	1,330	1,560
	2	150	1,310	1,530
	3	170	1,320	1,470
6-Syn. Greywater	1	370	960	1,270
	2	410	1,080	1,230
	3	360	1,170	1,370

Table A7. Field cast specimens of CSA-concrete unconfined compressive strength results.

Crater No.	Rep	Compressive Strength (psi) by Age (hr)		
		2 hr	24 hr	168 hr
1-Control	1	5,500	6,650	7,820
	2	5,730	7,000	6,920
	3	5,760	6,950	7,790
2-Pond	1	5,350	7,170	7,640
	2	5,600	6,850	7,780
	3	5,590	7,250	7,410
3-River	1	5,350	4,770	7,290
	2	5,290	5,950	7,010
	3	5,270	6,400	6,830
4-Brackish	1	5,170	5,780	7,390
	2	4,810	6,450	5,990
	3	4,430	6,690	5,940
5-Seawater	1	3,700	6,130	7,440
	2	3,950	6,380	7,010
	3	4,370	6,220	7,460
6-Syn. Greywater	1	4,750	6,140	6,530
	2	4,870	6,300	7,170
	3	5,050	6,300	6,640

Table A8. Field cores of CSA-concrete unconfined compressive strength and density.

Crater No.	Age (days)	Rep	Compressive Strength (psi)	Density (lb/ft ³)
1-Control	160	1	10,310	145.4
		2	11,300	146.3
		3	11,510	145.3
2-Pond	159	1	10,750	143.4
		2	9,930	142.9
		3	10,420	143.9
3-River	158	1	10,180	143.5
		2	11,150	144.0
		3	11,730	143.5
4-Brackish	157	1	10,300	143.5
		2	10,920	143.0
		3	10,770	143.6
5-Seawater	154	1	10,810	143.0
		2	9,040	142.8
		3	10,300	142.6
6-Syn. Greywater	153	1	9,950	139.8
		2	9,920	141.3
		3	9,450	142.8

Table A9. Trafficking visual inspection data at 1,024 cumulative passes.

Crater No.	Edge Distress	Location	Depth (in.)	Length (in.)	Width (in.)	FOD Size (in.)	Spall Failure?
1-Control	Spalling	W	0.25	91	3	3 x 2 x 0.75	No
	Spalling	E	0.125	67	1	0.25	No
	Edge Cracking	N		95			No
	Edge Cracking	S		92.5			No
2-Pond	Spalling	W	0.75	81.2	3.5	4 x 2 x 0.75	No
	Spalling	E	0.125	56	1	0.75	No
	Spalling	N	0.25	18	2	1	No
	Spalling	S	0.125	85	2	0.25	No
3-River	Spalling	W	0.75	92.5	2.5	1	No
	Spalling	E	0.75	93	2.25	2.5 x 2 x 0.75	No
	Edge Cracking	N		89			No
	Edge Cracking	S		92			No
4-Brackish	Spalling	W	0.75	90	3	2 x 2 x 0.25	No
	Spalling	E	0.875	89	2	1	No
	Edge Cracking	N		92			No
	Spalling	S	0.5	23	1.75	0.25	No
5-Seawater	Spalling	W	1	90	2.75	2.5 x 2 x 0.75	No
	Spalling	E	1	76	3	1 x 1 x 0.25	No
	Spalling	N	0.25	18	1.5	0.25	No
	Edge Cracking	S		92			No
6-Syn. Greywater	Spalling	W	0.25	60	2	0.25	No
	Spalling	E	0.25	83	1	0.25	No
	Spalling	N	0.25	15	2	0.25	No
	Edge Cracking	S		90			No

Table A10. Trafficking visual inspection data at 1,536 cumulative passes.

Crater No.	Edge Distress	Location	Depth (in.)	Length (in.)	Width (in.)	FOD Size (in.)	Spall Failure?
1-Control	Spalling	W	1.25	91	3	0.5	No
	Spalling	E	0.5	82	1	0.25	No
	Edge Cracking	N		95			No
	Edge Cracking	S		93			No
	Linear Cracking	N-S across slab		99			---
2-Pond	Spalling	W	0.75	82	3.5	0.5	No
	Spalling	E	0.25	85	2	0.25	No
	Spalling	N	0.5	36	2	0.25	No
	Spalling	S	0.5	85	2.25	0.25	No
3-River	Spalling	W	0.75	93	2.5	0.75	No
	Spalling	E	1	93	4.5	0.75	No
	Edge Cracking	N		90			No
	Spalling	S	0.25	24	2.5	0.25	No
4-Brackish	Spalling	W	0.75	91	3	0.5	No
	Spalling	E	1	92	2.5	0.25	No
	Edge Cracking	N		92			No
	Spalling	S	0.5	23	1.75	0.25	No
5-Seawater	Spalling	W	1.5	91	3.5	0.25	No
	Spalling	E	1	78	3	0.5	No
	Spalling	N	0.5	18	2	0.25	No
	Spalling	S	0.5	46	3	0.25	No
	Linear Cracking	N-S across slab		99			---
6-Syn. Greywater	Spalling	W	1	68	6	0.5	No
	Spalling	E	1	90	2.75	0.25	No
	Spalling	N	0.25	15	2	0.25	No
	Edge Cracking	S		91			No

Table A11. Trafficking visual inspection data at 2,048 cumulative passes.

Crater No.	Edge Distress	Location	Depth (in.)	Length (in.)	Width (in.)	FOD Size (in.)	Spall Failure?
1-Control	Spalling	W	1	91	3	0.25	No
	Spalling	E	0.5	90	1.5	0.25	No
	Edge Cracking	N		95			No
	Spalling	S	0.25	93	1	0.25	No
2-Pond	Spalling	W	0.75	90	3.5	0.25	No
	Spalling	E	0.5	91	2	0.25	No
	Spalling	N	0.25	42	2	0.25	No
	Spalling	S	0.5	90	2.25	0.25	No
3-River	Spalling	W	1	93	2.5	0.25	No
	Spalling	E	1.5	94	4.5	0.25	No
	Spalling	N	0.25	90	1	0.125	No
	Spalling	S	0.25	89	2.5	0.25	No
4-Brackish	Spalling	W	1	92	5.5	0.25	No
	Spalling	E	1.25	92	2.2	0.25	No
	Edge Cracking	N		93			No
	Spalling	S	0.5	90	2	0.25	No
5-Seawater	Spalling	W	1.75	92	3.5	0.25	No
	Spalling	E	1	90	4	0.25	No
	Spalling	N	0.5	18	2	0.25	No
	Spalling	S	0.5	48	3	0.5	No
6-Syn. Greywater	Spalling	W	1	78	8	0.75	No
	Spalling	E	1	90	2.75	0.25	No
	Spalling	N	0.25	15	2	0.25	No
	Edge Cracking	S		91			No

Table A12. Trafficking visual inspection data at 2,560 cumulative passes.

Crater No.	Edge Distress	Location	Depth (in.)	Length (in.)	Width (in.)	FOD Size (in.)	Spall Failure?
1-Control	Spalling	W	1	91	3	0.25	No
	Spalling	E	1	90	1.5	0.25	No
	Edge Cracking	N		95			No
	Spalling	S	0.25	13	1	0.25	No
2-Pond	Spalling	W	0.75	90	3.5	0.25	No
	Spalling	E	0.5	91	2	0.25	No
	Spalling	N	0.25	42	3	0.25	No
	Spalling	S	0.5	90	2.25	0.25	No
3-River	Spalling	W	1	93	2.5	0.25	No
	Spalling	E	1.5	94	4.5	0.25	No
	Spalling	N	0.25	90	1	0.25	No
	Spalling	S	0.25	89	2.5	0.25	No
4-Brackish	Spalling	W	1	92	5.5	0.25	No
	Spalling	E	0.25	92	2.5	0.25	No
	Edge Cracking	N		93			No
	Spalling	S	0.5	90	2	0.25	No
5-Seawater	Spalling	W	1.75	92	3.5	0.25	No
	Spalling	E	1	90	4	0.25	No
	Spalling	N	0.5	18	2	0.25	No
	Spalling	S	0.5	46	3	0.5	No
6-Syn. Greywater	Spalling	W	1	78	8	0.75	No
	Spalling	E	1	90	2.75	0.25	No
	Spalling	N	0.25	15	2	0.25	No
	Edge Cracking	S		91			No

Table A13. Trafficking visual inspection data at 3,072 cumulative passes.

Crater No.	Edge Distress	Location	Depth (in.)	Length (in.)	Width (in.)	FOD Size (in.)	Spall Failure?
1-Control	Spalling	W	1.75	91	5	0.5	No
	Spalling	E	1.25	90	3	0.25	No
	Edge Cracking	N		95			No
	Spalling	S	0.25	13	1	0.25	No
2-Pond	Spalling	W	1.5	90	3.5	0.75	No
	Spalling	E	1	91	5	0.25	No
	Spalling	N	1.5	90	4	0.25	No
	Spalling	S	0.5	90	2.25	0.25	No
3-River	Spalling	W	1.5	93	10	0.75	No
	Spalling	E	2.25	94	4.5	0.25	No
	Spalling	N	0.25	90	1	0.25	No
	Spalling	S	0.75	90	4	0.25	No
4-Brackish	Spalling	W	1.5	92	8	0.5	No
	Spalling	E	1.5	92	3	0.5	No
	Edge Cracking	N		93			No
	Spalling	S	0.5	90	2	0.25	No
5-Seawater	Spalling	W	2.25	92	4	0.5	No
	Spalling	E	1.5	90	4	0.25	No
	Spalling	N	0.5	18	2	0.25	No
	Spalling	S	0.5	91	3	0.25	No
6-Syn. Greywater	Spalling	W	1.5	89	8	0.5	No
	Spalling	E	1.25	90	4	0.5	No
	Edge Cracking	N		90			No
	Edge Cracking	S	0.25	91			No

Table A14. Trafficking visual inspection data at 3,584 cumulative passes.

Crater No.	Edge Distress	Location	Depth (in.)	Length (in.)	Width (in.)	FOD Size (in.)	Spall Failure?
1-Control	Spalling	W	1.75	91	5	0.5	No
	Spalling	E	1.75	91	4.5	0.25	No
	Spalling	N	0.25	7	1.5	0.25	No
	Spalling	S	0.25	13	1	0.25	No
2-Pond	Spalling	W	1.5	91	3.5	0.25	No
	Spalling	E	1	91	5	0.25	No
	Spalling	N	1.5	90	4	0.25	No
	Spalling	S	0.5	90	2.25	0.25	No
3-River	Spalling	W	1.75	93	9	0.5	No
	Spalling	E	2.25	94	5	0.25	No
	Spalling	N	0.25	90	1	0.25	No
	Spalling	S	0.75	90	4	0.5	No
4-Brackish	Spalling	W	1.5	92	9	0.5	No
	Spalling	E	1.5	92	4.5	0.25	No
	Spalling	N	0.25	4	1	0.25	No
	Spalling	S	0.5	90	2	0.25	No
5-Seawater	Spalling	W	2.25	92	4	0.25	No
	Spalling	E	1.5	90	4	0.25	No
	Spalling	N	0.5	18	2	0.25	No
	Spalling	S	0.5	91	3	0.25	No
6-Syn. Greywater	Spalling	W	1.5	91	8	0.25	No
	Spalling	E	1.25	90	4	0.5	No
	Edge Cracking	N		90			No
	Spalling	S	0.25	91	1	0.25	No

Acronyms and Abbreviations

Term	Definition
ADR	Airfield Damage Repair
AFCEC	Air Force Civil Engineer Center
ANOVA	Analysis of Variation
CBR	California Bearing Ratio
CSA	Calcium Sulfoaluminate
CSAB	Calcium Sulfoaluminate Belite
DCP	Dynamic Cone Penetrometer
IC	Ion Chromatography
ICP	Inductively Coupled Plasma
OPC	Ordinary Portland Cement
RADR	Rapid Airfield Damage Recovery
ROWPU	Reverse Osmosis Water Purification Units
S	Synthetic Water Solutions
SM	Standard Method
S(S)	Synthetic Seawater without heavy metals
TOC	Total Organic Carbon
USAF	United States Air Force
XRD	X-Ray Diffraction
XRF	X-Ray Fluorescence

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic inches	1.6387064 E-05	cubic meters
cubic yards	0.7645549	cubic meters
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
feet	0.3048	meters
foot-pounds force	1.355818	joules
gallons (U.S. liquid)	3.785412 E-03	cubic meters
inches	0.0254	meters
ounces (mass)	0.02834952	kilograms
ounces (U.S. fluid)	2.957353 E-05	cubic meters
pints (U.S. liquid)	0.473176	liters
pounds (force)	4.448222	newtons
pounds (force) per inch	175.1268	newtons per meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.45359237	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
pounds (mass) per cubic inch	2.757990 E+04	kilograms per cubic meter
pounds (mass) per square foot	4.882428	kilograms per square meter
pounds (mass) per square yard	0.542492	kilograms per square meter
quarts (U.S. liquid)	9.463529 E-04	cubic meters
slugs	14.59390	kilograms
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters
square yards	0.8361274	square meters
yards	0.9144	meters

REPORT DOCUMENTATION PAGE

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14. ABSTRACT In austere environments with limited access to clean water, it is advantageous to use nonpotable water for construction (i.e., mixing water for concrete.) In rapid-response situations such as rapid airfield damage recovery (RADR), the use of calcium sulfoaluminate (CSA) cements is beneficial for expedient pavement repairs because of their rapid strength gain characteristics. However, the hydration products formed by CSA cements are substantially different from those formed by ordinary portland cement and might react differently to impurities that water sources may contain. A laboratory study component investigated the application of various salts and impure sources of mixing water with commercially available CSA cement-based products. A field component studied the application of naturally occurring impure water sources for RADR. Recommendations are made for implementation of impure mixing water for RADR using commercially available flowable fill and concrete products made with CSA cement.					
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