**US Army Corps
of Engineers®**
Engineer Research and
Development Center

Long-Term Durability of Cold Weather Concrete Phase I Report

Danielle E. Kennedy, Benjamin E. Watts, Charles E. Smith,
and Jared I. Oren

October 2019

The U.S. Army Engineer Research and Development Center (ERDC) solves the nation's toughest engineering and environmental challenges. ERDC develops innovative solutions in civil and military engineering, geospatial sciences, water resources, and environmental sciences for the Army, the Department of Defense, civilian agencies, and our nation's public good. Find out more at www.erdcl.usace.army.mil.

To search for other technical reports published by ERDC, visit the ERDC online library at <http://acwc.sdp.sirsi.net/client/default>.

Long-Term Durability of Cold Weather Concrete Phase I Report

Danielle E. Kennedy, Benjamin E. Watts, Charles E. Smith,
and Jared I. Oren

*U.S. Army Engineer Research and Development Center (ERDC)
Cold Regions Research and Engineering Laboratory (CRREL)
72 Lyme Road
Hanover, NH 03755-1290*

Final Report

Approved for public release; distribution is unlimited.

Prepared for Headquarters, U.S. Army Corps of Engineers
Washington, DC 20314-1000

Under PE 62784/T53, Innovative Construction Materials for the Arctic Program

Abstract

The placement of conventional concrete in sub-freezing temperatures necessitates costly, labor-intensive measures to prevent freezing and ensure adequate early strength. An alternative method of sub-freezing concrete construction employs antifreeze admixtures which concurrently depress the freezing point and accelerate the hydration of fresh concrete. This concrete mixture has been applied successfully in cold regions where there are sub-freezing temperatures, ultimately enabling an extension of the construction season. In the last two decades, multiple field demonstrations of this technology have been implemented, many of which remain in service. The materials and mixtures used at these sites were evaluated in the short term with laboratory testing; however, most have not been inspected over time to evaluate their long-term durability in service. This document includes characteristics of durability, such as nondestructive and destructive testing measures to determine durability under long-term field conditions.

DISCLAIMER: The contents of this report are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products. All product names and trademarks cited are the property of their respective owners. The findings of this report are not to be construed as an official Department of the Army position unless so designated by other authorized documents.

DESTROY THIS REPORT WHEN NO LONGER NEEDED. DO NOT RETURN IT TO THE ORIGINATOR.

Contents

Abstract	ii
Figures and Tables	v
Preface.....	vi
Unit Conversion Factors	vii
Acronyms and Abbreviations	viii
1 Introduction.....	1
1.1 Background	1
1.2 Research needs	1
1.3 Military needs	2
1.4 Objectives	3
1.5 Approach	3
1.6 Scope	3
2 Literature Review	4
2.1 Factors of durability	4
2.2 Salt induced corrosion of reinforcing steel.....	4
2.3 Freeze thaw durability	4
2.4 Durability Testing.....	6
2.4.1 Destructive testing.....	6
2.4.2 Nondestructive testing.....	7
2.5 Review of admixtures used in CWAS.....	8
2.5.1 Accelerating/antifreeze admixture chemistry.....	8
3 Review of CWAS development and Implementation.....	12
3.1 Laboratory screening and selection of CWAS	12
3.1.1 Plastic properties.....	14
3.1.2 Slump loss.....	14
3.1.3 Air content	15
3.1.4 Initial freeze point.....	15
3.1.5 Compressive strength.....	15
3.1.6 Freeze-thaw durability.....	15
3.1.7 Time of set.....	16
3.2 Field implementation of CWAS	17
3.2.1 Sault Ste. Marie, Michigan.....	17
3.2.2 Littleton, New Hampshire.....	18
3.2.3 Rhinelander, Wisconsin	19
3.2.4 North Woodstock, New Hampshire.....	20
3.2.5 West Lebanon, New Hampshire	21
3.2.6 Concord, New Hampshire.....	22
3.2.7 Grand Forks Air Force Base, North Dakota.....	23

3.2.8 Fort Wainwright, AK.....	25
3.2.9 Fort Drum, New York.....	26
3.2.10 Airfield damage repair.....	27
4 Follow-on Field Site Investigations.....	28
4.1 Equipment and test descriptions	28
4.1.1 Ultrasonic pulse velocity (UPV).....	28
4.1.2 Ground penetrating radar (GPR).....	28
4.1.3 Light weight deflectometer (LWD)	29
4.1.4 Coring.....	29
4.2 Field Site Investigations	29
4.2.1 Sault Ste. Marie, MI	29
4.2.2 Fort Wainwright, AK.....	33
4.2.3 Fairbanks, AK.....	36
4.2.4 West Lebanon, NH.....	38
4.2.5 Treat Island, Maine	42
4.3 Discussion of results	43
4.3.1 Sault Ste. Marie, Michigan.....	43
4.3.2 Fort Wainwright, Alaska	43
4.3.3 Fairbanks, Alaska.....	44
4.3.4 West Lebanon, New Hampshire	44
5 Conclusions and Recommendations.....	45
5.1 Conclusions	45
5.2 Field specimen analysis.....	45
5.3 Laboratory testing.....	47
5.4 Guidance and tools for implementation of CWAS	48
References	50

Report Documentation Page

Figures and Tables

Figures

Figure 1. Sault Ste. Marie, MI Slab 2 (control).	31
Figure 2. Sault Ste. Marie, MI Slab 3.	32
Figure 3. A–C Nondestructive Testing Methods, GPR, LWD, and UPV, respectively.	33
Figure 4. Fort Wainwright, AK, site in 1 July 2018.	34
Figure 5. Fort Wainwright, AK, Slab 3.	36
Figure 6. CCHRC Slab 1 July 2018.	38
Figure 7. West Lebanon, NH, control concrete.	40
Figure 8. West Lebanon, NH, CWAS concrete.	40
Figure 9. West Lebanon, NH, control concrete.	41
Figure 10. West Lebanon, NH, CWAS concrete.	41

Tables

Table 1. Summary of accelerating/antifreeze admixture chemistry for the (Korhonen et al. 2004) report.	11
Table 2. Grace mixture proportions.	13
Table 3. Master Builders Inc. mixture proportions.	14
Table 4. Time of set data.	16
Table 5. Mixture properties of Soo Locks, MI, site.	18
Table 6. Hardened core properties of Soo Locks, MI, site.	18
Table 7. Fresh concrete properties of Littleton, NH, project.	19
Table 8. Strength values estimated for Littleton, NH.	19
Table 9. Fresh concrete properties of Rhinelander, WI, project.	20
Table 10. Plastic properties of North Woodstock, NH, project.	21
Table 11. Plastic properties of West Lebanon, NH, project.	22
Table 12. Plastic properties of Grand Forks, ND, USAF project.	24
Table 13. Compressive strength and maturity index data from cylinders cured 24 hours in the field.	25
Table 14. Plastic properties for each slab of Fort Wainwright, AK, project.	26
Table 15. Sault Ste. Marie, MI, mixture proportions.	30
Table 16. Fort Wainwright, AK, mixture proportions.	35
Table 17. CCHRC mixture proportions.	37
Table 18. West Lebanon, NH, CWAS concrete mixture proportions.	39

Preface

This study was conducted for the U.S. Army Corps of Engineers under PE 62784/T53, Innovative Construction Materials for the Arctic Program. The technical monitor was Mr. Josh Fairley (CEERD-GMM).

The work was performed by the Engineering Resources Branch (CEERD-RRE) of the Research and Engineering Division (CEERD-RR), U.S. Army Engineer Research and Development Center, Cold Regions Research and Engineering Laboratory (ERDC-CRREL). At the time of publication, Mr. Jared L. Oren was acting Division Chief, CEERD-RR; and Dr. Caitlin Callaghan was acting Chief, CEERD-RRE. The Deputy Director of ERDC-CRREL was Mr. David B. Ringelberg and the Director was Dr. Joseph L. Corriveau.

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

Unit Conversion Factors

Multiply	By	To Obtain
acres	4,046.873	square meters
acre-feet	1,233.5	cubic meters
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
inches	0.0254	meters
inch-pounds (force)	0.1129848	newton meters
miles (U.S. statute)	1,609.347	meters
pounds (force)	4.448222	newtons
pounds (force) per foot	14.59390	newtons per meter
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
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters
square miles	2.589998 E+06	square meters
square yards	0.8361274	square meters
tons (force)	8,896.443	newtons
tons (force) per square foot	95.76052	kilopascals
tons (long) per cubic yard	1,328.939	kilograms per cubic meter
yards	0.9144	meters

Acronyms and Abbreviations

Acronym	Meaning
ACI	American Concrete Institute
ASR	Alkali-Silica Reaction
ATR	Adiabatic Temperature Rise
CCHRC	Cold Climate Housing Research Center
CRREL	Cold Regions Research and Engineering Laboratory
CWAS	Cold Weather Admixture Systems
cwt	Hundredweight
D-cracking	Durability Cracking
DoD	Department of Defense
DTA	Differential Thermal Analysis
ERDC	Engineer Research and Development Center
FWD	Falling Weight Deflectometer
GPR	Ground Penetrating Radar
LWD	Light Weight Deflectometer
MOE	Modulus of Elasticity
MSDS	Materials Safety Data Sheets
NHDOT	New Hampshire Department of Transportation
pcy	pounds per cubic yard
RCC	Roller-Compacted Concrete
RDM	Relative Dynamic Modulus
SCM	Supplementary Cementitious Materials
UPV	Ultrasonic Pulse Velocity
USACE	U.S. Army Corps of Engineers
USAF	U.S. Air Force
w/c	water to cement ratio

1 Introduction

1.1 Background

Ambient temperatures below the freezing point of water present significant challenges to concrete placement. Traditional freeze protection measures may require the subgrade or base to be thawed and heated, including heating the mixing water and concrete mixture before placement. Following placement, the concrete must be tented and thermally insulated during initial curing; this allows for adequate strength gain. Ease of construction is also impacted; workers report physical constraints due to working in a tent, which does not allow free movement for finishing concrete (Korhonen et al. 2004). With these limitations in mind, the U.S. Army Corps of Engineers (USACE), Cold Regions Research and Engineering Laboratory (CRREL) began work in the early 1990s to develop an alternative technology for placement of concrete in sub-freezing conditions which did not require costly freeze protection measures. This technology employs Cold Weather Admixture Systems (CWAS) that incorporate off-the-shelf accelerating and antifreeze admixtures which accelerate the heat release associated with portland cement hydration. This enables the self-heating of fresh concrete to outpace the cooling effect of a sub-freezing environment.

Accelerating or “antifreeze” admixtures have been incorporated around the world since the 1950s (Korhonen 1990) to ensure that fresh concrete placed in low temperatures will not freeze at an early age; this condition causes low strength gain and poor durability. These admixtures prevent the freezing of concrete by accelerating the already exothermic hydration reaction of portland cement, increasing the rate of internal heat generation. Some accelerating admixtures also depress the freezing point of the fresh concrete, thereby providing additional protection from early freezing. While admixtures introduced into the concrete mixture are not free of cost, they are lower than the overall cost to construct a tent and heat the concrete area for the time required.

1.2 Research needs

Long-term durability data must be compiled from the known test sites in order to verify the durability of antifreeze concrete over long periods of

time in various parts of the country. Laboratory settings create an ideal environment for long-term durability testing of concrete resistance to freeze thaw. Field data must also be collected to correlate laboratory testing with the in-place durability of concrete placements in cold environments. Completing both destructive and nondestructive testing of field specimens will allow validation of both laboratory testing and the in-field construction to ensure long-term durability is achieved and possible.

In addition, laboratory testing of variability in the mix designs as far as admixtures, cement types, and addition of supplementary cementitious materials (SCM) are involved is lacking. Currently, no published CWAS project incorporates cementitious materials other than type I/II portland cement. SCM have been shown to mitigate alkali-silica reaction (ASR), reduce permeability, increase strength, and allow mixtures to be more economical.

The concrete elements constructed using cold weather concrete have been limited mostly to horizontal construction. The creation of vertical structures using cold weather concrete has not been developed and requires further research to validate. The development of durable cold weather concrete vertical structures will allow even more construction during a season that does not typically allow it. Extensive field testing will need to be completed to ensure the durability of these structures.

Further laboratory and field testing must be completed to verify the durability of cold weather concrete. Durability has been proven in laboratory testing and field sites have been constructed over the last decades. Investigation of these field sites is required to verify both the laboratory testing and that the field sites have remained durable over time.

1.3 Military needs

The Department of Defense (DoD) has identified a need to overcome engineering challenges related to infrastructure in cold regions (White House 2013). The Artic Strategy provides requirements in order to work collaboratively to ensure the artic is a “secure and stable region” (DoD 2016). The ability to construct concrete in sub-freezing temperatures allows for an extended construction season and more freedom of movement on the battlefield. This not only extends the timeframe that construction can be completed but also the ambient temperatures in which concrete can be placed. Cold weather concrete has been instrumental in forging a path for

construction in cold regions. This technology has been used for various construction projects in the United States and has the ability to be implemented in various scenarios where there are sub-freezing temperatures.

1.4 Objectives

The primary objective of this study is to assess the durability of in-place concrete elements constructed using the cold weather admixture systems technology developed at CRREL. Anecdotal reports from several field sites indicate that CWAS concrete performs well in service; this study aims to evaluate the validity of these reports through formal observation, collection of samples, and nondestructive testing.

1.5 Approach

The durability of existing concrete placements incorporating cold weather admixture systems will be evaluated on-site. Based on a review of previous CRREL reports on CWAS placements, researchers have identified several field sites ranging in age from 10 to 25 years. At each site, they will assess the durability of in-place concrete through visual evaluation and nondestructive testing. They will also collect core samples for purposes of future laboratory analysis.

1.6 Scope

The scope of this project includes inspection of various field sites where CWAS concrete has been placed and in service for over a decade. This includes travel to each site where nondestructive and destructive testing will be completed. Testing equipment consists of using a ground penetrating radar, lightweight deflectometer, ultrasonic pulse velocity meter, and a coring drill to collect data and/or cores from each site.

2 Literature Review

2.1 Factors of durability

Regions that experience significant periods of below freezing weather expose concrete to multiple potential failure mechanisms. These include damage from water freezing and thawing within the cementitious matrix of the concrete, degradation of the concrete from chemicals used to de-ice pavements, and corrosion of reinforcing steel induced by the migration of chloride ions through the cementitious microstructure to the surface of steel rebar. The measures required to protect concrete from these different modes of attack vary, but can be quantified using standardized test methods.

2.2 Salt induced corrosion of reinforcing steel

Chloride ions from road salt or seawater can accelerate the corrosion of reinforcing steel in concrete pavements and structures. The most effective method of preventing this is limiting the rate at which these ions can move through the concrete microstructure. This rate is known as diffusivity, and can be quantified using a number of different test methods (ASTM 2012b; ASTM 2012d; ASTM 2016b; ASTM 2012c). These methods all quantify properties of the cementitious microstructure which stem from the formation factor. This is defined as the ratio of the conductivity of a material to the conductivity of its pore solution (Archie 1942). This term effectively characterizes the pore structure of porous materials (Snyder 2001).

2.3 Freeze thaw durability

When water freezes, it expands in volume by approximately 9% (Monteiro 2006). The confined expansion of water within the cementitious matrix during freezing can result in cracking unless voids exist within the concrete for ice to form. This is accomplished by entraining air within the concrete. Entrained air consists of uniformly distributed cast-in air bubbles between 10 μm and 1000 μm in size, and is distinct from undesirable entrapped air, which occurs as large, irregular, non-spherical voids. The volume of air entrained within concrete is one of three parameters that define what is commonly referred to as the “air-void system.” The other two parameters, spacing factor and specific surface, respectively, quantify the average distance water must travel to reach an air-void and the number and size of entrained bubbles for a given volume

of air (Portland Cement Association 1998). Recommended values of spacing factor and specific surface for durable concrete are less than 0.2 mm and greater than 24 mm²/m³, respectively. While these values are known to correlate well with freeze thaw durability from studies of air-void systems in hardened concrete, they are also difficult to measure from fresh concrete in a construction environment. For this reason, the total air content is measured via the volumetric (ASTM 2016a) or pressure (ASTM 2017) methods. This is the most commonly specified value with ranges from 3–7.5%, based on severity of exposure and maximum aggregate size.

The presence of a proper air-void system in concrete is only effective for mitigating freeze-thaw damage when the voids provide empty space for ice to form. When the air void system in concrete becomes completely saturated, any protection that may have been provided is completely eliminated (Litvan 1988). The maximum degree to which the air void system in concrete can be saturated and still protect the concrete from freeze thaw damage is termed the critical degree of saturation, and has been experimentally determined to be approximately 85–90% (Li et al. 2011). Increasing the air content can increase the time required to saturate the air void system, but does not completely eliminate the potential for freeze-thaw damage. The rate at which water can be absorbed by concrete in a continuously saturated system can be slowed by densifying the microstructure or waterproofing the surface of the concrete. Proper drainage of concrete elements that are not continuously saturated is also a key factor.

Freezing and thawing is potentially the most destructive weathering factor, even more so with the presence of deicing chemicals. Aggregates with poor freeze-thaw durability can develop durability cracking, otherwise known as “D-cracking.” Damage through D-cracking begins with cracking in the aggregate and extends out to the paste. It will usually start at the bottom of the slab and propagate upward. Corners are most susceptible to damage because more water is available, causing more expansion.

Several years of laboratory testing ensured the durability of these antifreeze concrete mixtures and several field sites have been constructed. However, there have been very few cases of investigation after construction to ensure long-term durability in the field. Durability is defined in American Concrete Institute (ACI) concrete terminology as “the ability of a material to resist weathering action, chemical attack, abrasion,

and other conditions of service” (ACI 2016). Compressive strength, water to cement ratio (w/c), and air entrainment are typical measures of concrete durability, but not always the best or only way to do so.

While high compressive strength and a low w/c will provide a higher resistance to cracking and positively influence permeability, resistance to exposure conditions should also be taken into consideration, especially for cold weather concrete. Additional performance characteristics to determine sufficient durability are modulus of elasticity, creep, permeability, shrinkage, lamination, and more importantly for cold weather concrete, freeze-thaw resistance. Cold weather concrete has more restrictions than typical placements due to the inclusion of specific admixtures and lowered curing temperatures.

Other factors that are related to concrete durability, including freezing and thawing, are the following: high humidity and rain, ultraviolet resistance, chemical resistance, sulfate attack, seawater exposure, corrosion, alkali-silica reaction, and abrasion. Concrete can be destroyed over time by more than just time itself; these other factors apply when considering a structure or concrete element for long-term durability.

2.4 Durability Testing

2.4.1 Destructive testing

Currently, there are various standards and methods for destructive and nondestructive testing, including field and laboratory testing, which can determine if a concrete element will be or still is durable. The first, and arguably the most important for cold region concrete, is freeze-thaw testing. ASTM C666 (2015) was published to determine the resistance of concrete specimens to rapidly repeating cycles of freezing and thawing. This test is completed in the laboratory and does not provide a quantitative measure of life-of-service for a specific type of concrete.

For concrete either newly in place or previously placed, cylinder samples can be cast or cored for testing, respectively. Cylindrical samples are generally tested for modulus of elasticity, compressive strength, tensile strength, and air content. Modulus of elasticity samples made at the time of construction can be later compared to results from falling weight deflectometer values. Compressive strength is typically measured at the time of construction and up to 28 days later to ensure that the concrete

has met sufficient strength for the project. Cores can be taken from the area to measure the compressive or tensile strength after years in the field. Comparison of the control values from the initial placement to current day core strength can show if the concrete has sustained strength over time.

Air content is an important factor in regards to developing freeze-thaw resistant concrete. Air entraining admixtures are used to create small, well distributed bubbles in the cement paste, giving intruding water a place to go during freezing temperatures. Air content can be measured on the day of placement with fresh concrete through several test methods (ASTM 2016a; ASTM C231 2017). Air content can be measured in hardened specimens using the stereology method (ASTM C457 2016). Air content is an important factor, if it is too low the concrete will not be resistant to freeze-thaw damage; if it is too high, strength will suffer. If there is no data completed during the initial placement of the concrete and destruction is visible, stereology on the hardened concrete would show the air content of the hardened product and if it is sufficient for the mix design.

2.4.2 Nondestructive testing

Nondestructive testing can be completed on hardened concrete to determine durability. A falling weight deflectometer (FWD) can be used to measure vertical deflection by sending impulse load through pavement and measuring response (ASTM D4694 2009). The values can be used to estimate remaining structural capacity of a pavement and also determine required structural overlay depths. The modulus of elasticity (MOE) values estimated through use of the FWD can be compared to MOE values measured during the initial testing of cores, if the data was collected. In addition, the FWD can be used to measure deflection at two different points in time in order to see the degradation over time. This can assist with estimating the lifespan of the pavement.

The heavy weight deflectometer (HWD) operates similarly to the FWD and is used to assess bearing capacity of airfield pavements. Loads are applied to simulate a rolling aircraft in order to determine deflections. Using a ground penetrating radar (GPR) allows investigation of the subsurface (ASTM D6432, 2011). This can be used to view changes in material properties as well as voids and cracks.

Delamination caused by corrosion can be detected through dragged chains or a hammer, according to ASTM D4580 (2012). Corrosion can induce

cracking and cause the concrete cover to separate from the concrete substrate, so it is important to determine if it has occurred in a reinforced concrete element when testing for long-term durability. A GPR can also be used to image the subsurface and view changes in materials and cracks. Corrosion occurs in concrete multiple ways, 1) dissimilar metals such as steel reinforcement and aluminum conduit pipes, and 2) differences in concentrations of dissolved ions. The difference in concentrations can occur due to alkalis, chlorides, or oxygen. Cracking in the concrete can allow moisture and salts to reach the metal reinforcement and cause corrosion.

2.5 Review of admixtures used in CWAS

Previous work at CRREL focused on evaluating and selecting optimum combinations of antifreeze and accelerating admixtures with traditional water-reducing and air-entraining admixtures for use in self-heating cold weather concrete applications. A general review of antifreeze admixtures was conducted as part of Korhonen (1990); and several field sites supported the development of proprietary antifreeze/accelerating admixture systems for sub-freezing conditions (Korhonen and Brook 1996; Korhonen et al. 1997b; Korhonen et al. 1997a). However, the first unified approach to commercially available cold weather admixture systems was performed as part of Korhonen et al. 2004. This study evaluated combinations of water reducers, air entrainers, and antifreeze/accelerating admixtures from two commercial manufacturers. While the active chemical species from the antifreeze/accelerating admixtures used were not disclosed in their respective technical data sheets or in the report, examination of the materials safety data sheets (MSDS) enabled identification and approximate quantification of several chemicals known to act as accelerators in cementitious systems. The chemical constituents, ASTM C494 types, and brand names of the admixtures used in field sites from 2001 onward are summarized in Table 1.

2.5.1 Accelerating/antifreeze admixture chemistry

The two most abundant chemicals present in the admixtures summarized in Table 1 are calcium nitrate and calcium nitrite, both of which act as accelerators in portland cement-based systems. Calcium nitrate, commonly known as saltpeter, is a common constituent of accelerating admixtures designated for use in reinforced concrete systems. The most effective accelerating admixture, calcium chloride, is not suitable for

concrete with embedded steel due to the corrosion accelerating effects of chloride ions (Neville 1995). Calcium nitrate is known to have a benign and potentially inhibitive effect on steel corrosion (Dong et al. 2018), admixtures containing calcium nitrate are frequently marketed as “non-chloride” accelerators for this reason. Calcium nitrite has a similar accelerating effect; however, the nitrite ions provide significant and well characterized corrosion protection for embedded steel (Berke and Hicks 2004). Most calcium-nitrite admixtures are primarily marketed as corrosion inhibitors.

A key difference between calcium nitrate or calcium nitrite accelerators and other types of admixtures, is the dosage required for effectiveness. Most admixtures (water-reducers, air-entrainers, retarders) are dosed in fluid ounces per hundred pounds of cement (cwt); typical quantities are low-tens of ounces per cwt. Admixtures are normally packaged as aqueous solutions, and the mix water is adjusted to compensate for the relatively small amount of water contributed by the admixture. The relatively low dosage of admixture also results in very minor changes to w/c if additional admixture is added upon truck arrival. For calcium nitrate and calcium nitrite accelerators, the dosage is much higher, and can easily approach several gallons of admixture per cubic yard of concrete for normal (≈ 600 pounds per cubic yard (pcy)) cementitious contents. For a mixture with a w/c of 0.4 and an admixture with 40% calcium nitrite by mass, a typical dosage of 4 gallons pcy results in an effective increase from 0.4 to 0.44 w/c. This is not an issue if accounted for in the mixture design; however, accelerators are frequently added upon truck arrival, and the low w/c required at the batch plant to maintain an acceptable w/c after accelerator addition has in the past required additional water reducer (Korhonen et al. 2004). This complicating factor has important cost and staging implications.

Other chemicals present in Table 1 have either secondary accelerating effects (sodium thiocyanate, diethylene glycol, methylene diethanolamine) or function as water reducers (naphthalene sulfonate). The role of these secondary compounds relative to the more abundant calcium nitrate and calcium nitrite has not been investigated in previous research.

2.5.1.1 Freezing point depression of fresh concrete

A major component of previous work with cold weather admixture systems has been protection of fresh concrete from freezing through depression of the freezing point of the mixture. This has been largely

successful, the admixture systems used in field sites (Korhonen et al. 2004) effectively lowered the freezing point of the fresh concrete by 4–5°C ($\approx 23^{\circ}\text{F}$). An approximately linear relationship was observed between the solids contributed by the admixtures to the mix water and the amount of freezing point depression (Korhonen et al. 2004; Korhonen 2006). Because of the relatively high solids contents ($>10\%$) required to reduce the freezing point, the only realistic contributors to this are the calcium nitrate and nitrite admixtures, which require dramatically higher dosages than other types of admixtures or accelerators as previously mentioned.

Previous research of fresh concrete freezing depression was primarily for the protection from freezing during the first 24–48 hours of cold weather curing. While a lower freezing point theoretically reduces the freezing temperature of concrete at early ages, the acceleration of the exothermic portland cement hydration reaction is what actually prevents concrete elements from freezing in the field. This is confirmed by the temperature history profiles provided in several previous reports (Korhonen et al. 2004; Korhonen 2006). A largely unexplored topic is the longevity of the freezing point depression effect in hardened concrete. One study (Korhonen et al. 1998) examined the freezing point of concrete over time, but only out to 30 hours of age. During the first 30 hours of curing, the freezing point of the CWAS mixture dropped from -3.3°C to -8.4°C (26°F to 16°F). Examination of the cooling curves for each measured age shows a progressive reduction of the change in curvature associated with freezing. The duration of the freezing point depression effects associated with CWAS beyond 30 hours of age is currently unknown. Determination of the length of time for which this effect persists will be a focus of future research efforts.

Table 1. Summary of accelerating/antifreeze admixture chemistry for the (Korhonen et al. 2004) report.

Brand Name	Chemical Species	Action	ASTM Type	% Total Solids
POLARSET®	< 2% Calcium Bromide < 20% Calcium Nitrate < 20% Calcium Nitrite < 5% Diethylene Glycol < 2% Methylenediethanolamine	Cold weather corrosion inhibitor and accelerator	C	41.1
DCI® Corrosion Inhibitor	< 34% Calcium nitrite < 3% Calcium nitrate	Corrosion inhibitor and accelerator	C	33
DCI® S Corrosion Inhibitor	< 34% Calcium nitrite < 3% Calcium nitrate	Corrosion inhibitor and accelerator	–	33
Pozzutec® 20	< 40% Calcium Nitrate < 10% Naphthalenesulfonate < 5% Sodium Thiocyanate		C & E	47.63
Pozzutec® 20+	< 40% Calcium Nitrate < 5% Sodium Thiocyanate		C & E	47.63
Rheocrete® CNI	< 40% Calcium Nitrite	Corrosion inhibitor and accelerator	C	33

3 Review of CWAS development and Implementation

Antifreeze concrete admixtures have been incorporated into projects through the ERDC and CRREL for over twenty years. The following section will detail the known test sites and their respective laboratory and field studies. These field sites were constructed due to positive laboratory testing proving the mixtures could meet threshold design requirements. They had not been inspected to update their current state.

3.1 Laboratory screening and selection of CWAS

A 2004 laboratory study included a suite of testing to demonstrate how a chosen set of CWAS mixtures would perform at specified temperatures (Korhonen et al. 2004). Nine unique mixtures, eight admixture amended and one control, were developed for the experiment (shown in Table 2 and Table 3). The admixtures were procured from two vendors, Master Builders, Inc. (abbreviated MB, now known as BASF) and W.R. Grace and Co. (abbreviated WRG, now known as Grace Concrete Products). These mixtures were tested for both plastic and hardened properties to determine optimal mixtures for field sites placed in Littleton, NH, Rhinelander, WI, North Woodstock, NH, West Lebanon, NH, and Concord, NH.

Table 2. Grace mixture proportions.

Product		Admixture dosage				
		W.R. Grace & Co. suites (WRG)				
		I	II	III	IV	Control
Mira®70, mL/100 kg (fl oz/cwt)		780 (12)	585 (9)	390 (6)	130 (2)	130 (2)
ADVA® Flow, mL/100 kg (fl oz/cwt)		195 (3)	98 (1.5)	65 (1)	325 [65] (1)	-
DCI, L/m ³ (gal/yd ³)		30 (6)	--	30 (6)	-	-
DCI-S, L/m ³ (gal/yd ³)		--	30 (6)	--	30 (6)	--
PolarSet, L/100 kg (gal/cwt)		6.52 (0.78)	6.52 (0.78)	6.52 (0.78)	3.26 (0.39) [2.61] (0.31)	-
Daratard® 17, mL/100 kg (fl oz/cwt)		-	-	-	260 (4)	-
Eclipse® Plus (% cement wt)		-	-	1%	-	-
Darex® II AEA (mL/100 kg)		60 (0.9) [30] (0.45)	30 (0.45) [45] (0.68)	20 (0.3)	60 (0.9) [30] (0.45)	25 (0.38)
w/c ratio (w/o admix)	1	0.317	0.329	0.321	0.303	0.435
w/c ratio (w/ admix)	2	0.390	0.400	0.390	0.400	0.436
	3	[0.442]	[0.452]	[0.442]	[0.421]	N A

Table 3. Master Builders Inc. mixture proportions.

Product		Admixture dosage ([Brackets] indicate additional dosage or condition at jobsite)				
		Master Builders, Inc. suites (MB)				
		I	II	III	IV	Control
Polyheed®997, mL/100 kg (fl oz/cwt)		780 (12)	780 (12)	390 (6)	780 (12)	130 (2)
Glenium®3000, NS mL/100 kg (fl oz/cwt)		195 (3)	195 (3)	65 (1)	-	-
Rheocrete®CNI, L/m ³ (gal/yd ³)		30 (6)	30 (6)	30 (6)	30 (6)	-
Pozzutech®20, L/100 kg (gal/yd ³)		5.87 (0.7)	5.87 (0.7)	5.87 (0.7)	-	-
Pozzutech®20+, mL/100 kg (fl oz/cwt)		-	-	-	5.87 (0.7)	-
Pozzolite®100-XR, mL/100 kg (fl oz/cwt)		-	65 (1)	-	-	-
Tetraguard®AS20, % cement weight		-	-	1%	-	-
MB-VR® Standard, mL/100 kg (fl oz/cwt)		40 (0.6)	20 (0.3)	60 (0.9)	20 (0.3)	25 (0.38)
		[20] (0.3)	[20] (0.3)	[60] (0.9)		
w/c ratio (w/o admix)	1	0.316	0.316	0.320	0.271	0.435
w/c ratio (w/ admix)	2	0.430	0.390	0.390	0.390	0.436
	3	[0.430]	[0.431]	[0.431]	N/A	N A

3.1.1 Plastic properties

In order to confirm the use of the chosen eight admixture amended mixtures, plastic property testing was completed. These tests consisted of slump loss, air entrainment, and freeze-point depression. These tests provide information on the workability and ability to withstand freezing temperatures through depressed freezing point and proper air void system.

3.1.2 Slump loss

In testing for slump loss, mixtures were tested in small batches using a small drum mixer. The slump of the nine individual mixtures was measured incrementally every 5–10 minutes after the first 15 minutes until reaching 110 minutes. It was estimated that at a field site, the first 60 minutes would be considered a transit and adjustment period while the working period would be in the 60–90 minute range. At 60 minutes, the

control was at 50 mm, while all admixture amended mixtures ranged between 50–205 mm. The target slump during the working period was 100 mm and the WRG I, WRG II, and MB II reached this.

3.1.3 Air content

Following slump loss, air content at two different times was measured on the nine mixtures. The target air content range was 4.5–7.5% using the volumetric method (ASTM 2016a). After five minutes of mixing, only WRG IV, Control, and MB II were within the target range. After 55 minutes of mixing, only Control and MB IV mixtures were within the specified range. Although not all mixtures were in the specified range, this testing showed that the suite of admixtures did not prevent air entrainment in the concrete.

3.1.4 Initial freeze point

The final test before moving onto hardened properties was measuring initial freezing point of the mortar mixtures. This test procedure included embedding thermocouples into the concrete mixtures and keeping them in a -20°C (-4°F) room. The temperature of the concrete vs. time curve showed a deviation in the plot which is the freezing point. Aside from the control, all eight admixture amended mixtures surpassed the target of -5°C (23°F). The control initial freeze point was -1°C (30°F).

3.1.5 Compressive strength

Compressive strength testing was completed following ASTM C39 (ASTM C39/C39M-18 2018) and results were recorded for six different ages, 1-day, 3-day, 7-day, 14-day, 28-day, and 56-day. The concrete remained in plastic molds with the lid on until they were tested. When cured at 25°C (77°F), all mixtures performed better than the control. The same trend followed through with samples cured in the 5°C (41°F) environment. At -4°C (25°F), no damage took place during curing, and 56-day strengths exceeded the control at 25°C at 28 days. In addition, all MB mixtures exceeded the 25°C (77°F) control at 28 days.

3.1.6 Freeze-thaw durability

Freeze-thaw testing was run on the concrete mixtures following ASTM C666 (2015). Of the four WRG and MB mixtures, two sets were run: one with air entrainment and one without. The beams were all cured for 28 days before

beginning the testing. The data from the publication shows that three of the four non-air entrained WRG mixtures performed similarly to the control when measuring relative dynamic modulus (RDM) over 300 freeze-thaw cycles. Incorporating air entrainment, the mixtures performed better and lasted longer before reaching 60% of RDM, the failure point of the test. The data shows that with proper air entrainment, MB and WRG mixtures will likely perform similarly or better than the control.

3.1.7 Time of set

Finally, time of set and freezing point were measured on the same nine mixtures. Mortar was mixed and placed into cylindrical molds, the molds were placed into a water bath to keep them at the same three test temperatures, 25°C (77°F), 5°C (41°F), and -4°C (25°F). A saltwater bath was used for the below freezing temperature. The results for initial setting time of each mixture at the three temperatures is shown in Table 4.

Table 4. Time of set data.

Admixture Suite	25 °C (77 °F)	5 °C (41 °F)	-4 °C (25 °F)
WRG I	2:57	5:53	11:38
WRG II	3:12	6:23	14:28
WRG III	2:57	5:40	11:51
WRG IV	-	-	24:33
Control	4:34	11:21	-
MB I	3:47	9:53	13:57
MB II	5:02	15:02	25:10
MB III	3:37	10:46	11:50
MB IV	3:19	4:20	6:20
Control	4:38	12:18	-

The goal of this experiment was to determine whether admixture amended mixtures at -4°C (25°F) would set in the same or less time than the control mortar at 5°C (41°F).

3.2 Field implementation of CWAS

3.2.1 Sault Ste. Marie, Michigan

Six reinforced slabs on grade were placed at the Soo Locks in Sault Ste. Marie, Michigan, in March of 1994 (Korhonen et al. 1997a). This placement was the first to intentionally place fresh concrete in below freezing temperatures without insulation or heating. Admixtures were used from WRG and MB as discussed previously. During this study, there was no standard for an antifreeze admixture, so the following criteria were used:

- Depress the freezing point of water
- Promote strength gain of concrete at low temperatures
- Not interfere with concrete strength gain at normal temperatures
- Maintain the workability of the concrete
- Achieve a reasonable concrete set time
- Produce freeze-thaw durable concrete
- Not react with silica aggregate
- Not corrode steel
- Not adversely alter hydration products
- Be cost effective.

Since this publication (Korhonen et al. 1997a), a test method has been created for the application of cold-weather admixture systems and stipulates tests with suitable materials (ASTM 2016c). This specification was developed to ensure that admixtures used for temperatures to -5°C (23°F) are adequate and give sufficient plastic and hardened properties of the concrete.

The goal for minimum temperature was -5°C (23°F), with future goals being as low as -20°C (-4°F). In addition, the concrete was expected to gain strength with at least the same rate as concrete cured normally at 5°C (41°F). Laboratory testing over the course of two years demonstrated that the admixtures chosen by the manufacturers were not detrimental to the concrete with regards to freeze-thaw durability and shrinkage. After laboratory testing was complete, a field evaluation of the Soo Locks began.

The field site consisted of six reinforced slabs on grade of 5.5 m wide by 6.1 m long by 15 cm thick (18 ft by 20 ft by 6 in.). The surrounding site was prepared by jackhammering and replacing 15 cm (6 in.) of base material

with coarse gravel and setting up the forms and reinforcing steel. A control section was placed and covered with a heated enclosure. The second section, which included antifreeze admixtures, was covered with an unheated enclosure. The final four sections, which also included antifreeze admixtures, had no enclosure and were exposed to ambient air temperatures.

The properties of each mixture are shown in Table 5. There were no complications with placing the slabs. The concrete was left thermally unprotected, although a plastic sheet was placed over the finished area outside to reduce moisture loss due to wind. Cylinders were cast from each mix and stored on grade next to the slabs as well as overhead in a heated enclosure. Unfortunately, the cylinders stored outside were damaged due to freezing, rendering them untestable. This was likely due to the relatively small size of the cylinders compared to the concrete slabs. The cylinders stored inside the enclosure were heat cured and not useful to prove strength gain at freezing temperatures. Due to these circumstances, cores were drilled the following summer and tested, the properties of which are shown in Table 6. Test sites completed in the future typically kept the cast cylinders in a nearby cooler to keep them from freezing and becoming unusable.

Table 5. Mixture properties of Soo Locks, MI, site.

Mix	Slump, mm (in.)	Air, %	Unit Weight, kg/m ³ (lb/ft ³)	Interior Temp, °C (°F)
Control	51 (2)	3.2	2,307 (144)	12.2 (54)
EY11 Low Dose	140 (5.5)	3.2	2,307 (144)	3.3 (38)
EY11 High Dose	140 (5.5)	4.7	2,275 (142)	3.3 (38)
DP	127 (5)	8	2,163 (135)	7.2 (45)

Table 6. Hardened core properties of Soo Locks, MI, site.

Mix	Compressive strength, MPa (psi)	Bulk Density, g/cm ³ (lb/ft ³)	Evidence of past ice?
Control	46.7 (6,770)	2.31 (144)	No
EY11 Low Dose	50.6 (7,340)	2.32 (145)	No
EY11 High Dose	55.5 (8,050)	2.29 (143)	No
DP	46.0 (6,670)	2.21 (138)	No

3.2.2 Littleton, New Hampshire

A bridge curb was replaced in Littleton, New Hampshire, in December of 2001 with the use of CWAS (Korhonen et al. 2004). This project, along

with the following four, was completed for the Phase I of a project titled: Extending the Season for Concrete Construction and Repair. These projects were in conjunction with the Federal Highway Administration (FHWA), CRREL, and the following ten state departments: Idaho, Michigan, Montana, New Hampshire, New York, Pennsylvania, Utah, Vermont, Wisconsin, and Wyoming.

The repaired area measured 43 cm wide by 12 m long by 25 cm deep (17 in. by 4.7 in. by 9.8 in.) and contained steel reinforcing bars with anchor bolts for the guardrail. The concrete was transported in from a ready-mix plant about 30 km (19 miles) away. The properties of the fresh concrete are shown in Table 7. Cylinders were cast at the time of placement and average temperature was verified to be -5.2°C (23°F). Strength gain was estimated by way of a maturity curve based on temperatures recorded during curing. The strengths and design strengths are shown in Table 8.

The cost of this project, if it were to involve tenting and heating instead of CWAS, was estimated to be \$2,500. This consisted of building the shelter and heating during placement and initial curing. However, the admixtures for the mixture cost under \$250, only 10% of the heating cost, with the total cost of the concrete estimated to be \$190. Although the admixtures cost more than the concrete, the overall project cost was significantly decreased through their use.

Table 7. Fresh concrete properties of Littleton, NH, project.

Unit Weight, kg/m ³ (lb/ft ³)	Slump, mm (in.)	Air Content, %	Temperature, °C (°F)
2,310 (144)	180 (7.1)	5.1	8 (46)

Table 8. Strength values estimated for Littleton, NH.

12 hours, MPa (psi)	24 hours, MPa (psi)	Design Strength, MPa (psi)
7 (1,020)	17 (2,470)	35 (5,080)

3.2.3 Rhinelander, Wisconsin

The next implementation of the antifreeze admixtures was a pavement section in Rhinelander, Wisconsin, placed in February 2002 (Korhonen et al. 2004). This project was time sensitive, it was required to be open to traffic within 48 hours. To start off the project, the old concrete was removed and holes for dowel bars were drilled in the remaining concrete.

Plastic properties of the concrete were tested and are shown in Table 9. The w/c proved to be higher than desired. This was attributed to either the free water in the aggregates, or the addition of water during pumping. Three cylinders were cast and measured for freezing point, which was -5°C (23°F) in 30 minutes. An additional eleven cylinders were cast to estimate strength of the pavement. Nine of these cylinders were stored in a cooler next to the pavement, the remaining two were stored in 23°C (73°F) water. The concrete was imbedded with thermocouples to estimate the temperature, and, therefore, strength gain at the center, edge, and corner of the pavement slab.

Table 9. Fresh concrete properties of Rhinelander, WI, project.

Slump, mm (in.)	Air Content, %	Temperature, $^{\circ}\text{C}$ ($^{\circ}\text{F}$)
250 (9.8)	8	18 (64)

Estimates of strength gain over time were developed, with the highest strength in the center of the slab and the lowest in the corner. After 18 hours, the center of the pavement section was at 28 MPa (4,100 psi) with the edge at approximately 23 MPa (3,340 psi) and the corner at 15 MPa (2,180 psi).

As required, the new pavement was open to traffic within approximately 48 hours of initial construction, during which no cracks or signs of distress were recorded. At the time of this project, it was expected that cold weather concreting could double the cost of summer placement. This is caused by the heat, temporary enclosures, and insulation needed. For this project, an increase of $\$130/\text{m}^3$ was incurred for the cost of the antifreeze admixtures.

Eighteen months after placement, the pavement was inspected. There were no signs of major deterioration, but shallow map cracks were visible on the surface. These could have been caused by poor finishing or having a higher w/c than desired. The w/c for this project was difficult to regulate due to residual snow on the aggregate piles.

3.2.4 North Woodstock, New Hampshire

The third portion of the FHWA funded project began in the winter of 2002. The project consisted of constructing a new footing for the widening of Gordon Pond Brook Bridge in North Woodstock, New Hampshire

(Korhonen et al. 2004). The area of the new footing was 1 m wide by 2 m long by 1 m deep (3.3 ft by 6.6 ft by 3.3 ft) and included steel reinforcing bars. This was placed on the stream bottom, so the surrounding area was pumped dry before construction began.

Plastic properties were tested during placement and values obtained are shown in Table 10. Three cylinders were also created and measured to have a freezing point of -4.4°C (24°F). Further investigation of the mix gave an actual w/c ratio of 0.37, well below the target of 0.50. This lower w/c explains the higher than desired air content and why the cylinders did not reach a freezing point of -5.0°C (23°F).

Table 10. Plastic properties of North Woodstock, NH, project.

Unit Weight, kg/m ³ (lb/ft ³)	Slump, mm (in.)	Air Content, %	Temperature, °C (°F)
2,240 (140)	200 (7.9)	10	9 (48)

Fifty cylinders were cast to estimate the compressive strength gain. Twenty-one were stored next to the bridge in coolers, with nine stored in the stream, the final twenty were stored in room temperature conditions. Cylinders were tested periodically at 5°C (41°F) at CRREL in order to produce a strength gain curve. The center of the constructed footing was at 20 MPa (2,900 psi) within 48 hours, while the coolest portion took about four days to reach the threshold strength. The forms stayed in place for at least seven days to keep the concrete from water exposure due to the low temperatures.

For this project, the admixtures were added on-site, which caused no trouble and allowed a workable time of about an hour. Snow covered aggregates resulted in poor control of the w/c, which may have influenced the air content. Overall the project was a success.

3.2.5 West Lebanon, New Hampshire

The next segment of the FHWA project consisted of constructing bridge curbing and abutment on Trues Brook Bridge in West Lebanon, New Hampshire, in December of 2002 (Korhonen et al. 2004). The area of the curbing was 460 mm wide by 32 m long by 380 mm deep (18 in. by 105 ft by 15 in.) and required two truckloads of concrete. This project was the coldest weather for the FHWA projects so far.

The plastic properties of each truck are shown in Table 11. Cylinders were created to determine the freezing point of -6.6°C (20°F), below the target of -5.0°C (23°F). Additional cylinders were cast to determine the strength gain of the concrete. The concrete was dumped from the truck into the forms and consolidated with an internal vibrator. The concrete was finished and covered with a sheet of plastic and an insulation blanket. However, the exposed ends of the steel form ties were not covered and cooled below the freezing point.

Table 11. Plastic properties of West Lebanon, NH, project.

	Unit Weight kg/m^3 (lb/ft ³)	Slump mm (in.)	Air Content %	Temperature $^{\circ}\text{C}$ ($^{\circ}\text{F}$)
Truck 1	2,240 (140)	200 (7.9)	11.1	10 (50)
Truck 2	2,280 (142)	180 (7.1)	10.4	11 (52)

Additional cylinders were cast to develop a maturity curve from strengths of cylinders cured on site in coolers and at CRREL. The middle and top surface of the concrete reached the target of 20 MPa (2,900 psi) within three days, the coolest portion of the concrete took five days to reach this strength. After adequate strength gain, the forms were removed on day five.

The admixtures were not added to the concrete mix until the truck was at the jobsite, which allowed a longer time of workable concrete. Delayed addition allowed for lower water and admixture use, better control of slump, and lower cost overall. Although the concrete adjacent to the project froze, no serious problems were incurred. The cost of the admixtures for this project were approximately \$700. The cost to construct temporary enclosures for heating was \$750 for 100 labor-hours.

3.2.6 Concord, New Hampshire

The final segment of the FHWA project consisted of an 18 m (59 ft) segment of a 70 m (230 ft) long concrete sidewalk in Concord, New Hampshire, in February 2003 (Korhonen et al. 2004). The segment required additional antifreeze concrete through the exit (7 m, 23ft) and entrance ramps (11 m, 36 ft). The area of the sidewalk was 150 cm wide by 13 cm deep (5 ft by 5.1 in.) and consisted of welded-wire mesh reinforcement.

Plastic properties for the sidewalk were not recorded, although the freezing point of the first section of concrete was tested to be -6.5°C (20°F) and the w/c was retroactively calculated to be 0.34. The concrete was very stiff and adjustments were made using admixtures and water. The concrete was difficult to finish, but consolidated adequately with an internal vibrator. After finishing, the sidewalk was covered with a plastic sheet and insulation. Strength gain was estimated through the casting of cylinders and it took approximately 4.5 days for the warmest location to achieve 20 MPa (2,900 psi) and nearly 7.5 days for the coolest portion. All remaining forms were removed ten days after placement.

The overall goal of this project was to determine if it was acceptable to give the technology to another entity, the New Hampshire Department of Transportation (NHDOT) for immediate field use. Instructions were developed and the concrete was ordered, delivered, and admixtures dosed. Although there were complications with the slump initially, this project proved that the technology could be passed for immediate field use.

3.2.7 Grand Forks Air Force Base, North Dakota

This project completed on Grand Forks Air Force Base in North Dakota, was the first airfield pavement application developed with antifreeze concrete (Korhonen and Semen 2005). The project consisted of constructing an unreinforced parking apron in February 2004. The replacement of the 610 mm (24 in.) deep pavement was completed in conjunction with CRREL and the U.S. Air Force (USAF) 319th Civil Engineering Squadron.

Trial batches were created to ensure workability and that w/c would be below the 0.45 permitted for airfield pavements. The first trial batch had to be discarded due to a w/c of 0.497, slump of 6.5 mm (0.25 in.), freezing point of -4.3°C (24°F), and air content of 3%. The target slump and air content for the mix design were 150 mm (5.9 in.) and 6–8%, respectively. The second trial batch had slightly improved values with 273 mm (10.7 in.) slump, 11% air, and -4.7°C (24°F) freezing point. A third trial batch was not mixed, but adjustments were made to the previous batch to make it acceptable. Water content was reduced to lower slump and improve freezing temperature.

Thermocouples were put in place at the center, edge, and corner. Four truckloads were used for the entirety of the project. Due to the distance

from the ready mix plant to the site, most admixtures were added at the site. The plastic properties of each truckload are shown in Table 12. After dumping from the truck, the concrete was consolidated with an internal vibrator and leveled with a vibratory screed. The slab was covered with insulation to reduce evaporation on the windy day. Cylinders were cast and taken to the lab to test for strength gain.

Table 12. Plastic properties of Grand Forks, ND, USAF project.

Property	Target	Truck 1	Truck 2	Truck 3	Truck 4
Slump, mm (in.)	150 (6)	178 (7)	70 (3)	203 (8)	125 (5)
Air, %	6-8	12.25	6.8	9	–
Temperature, °C (°F)	10-20 (50-68)	17 (63)	20 (68)	20 (68)	11 (52)
Unit Weight, kg/m ³ (lb/ft ³)	–	2,191 (137)	2,287 (143)	2,243 (140)	–
Freezing Point, °C (°F)	-5 (23)	-7 (20)	-7 (19)	-8 (18)	-6 (22)

The compressive strength gain is shown in Table 13. This pavement had the strength potential to be opened to traffic after approximately 48 hours. The goal of this project was to resist freezing down to -5°C (23°F); the temperatures were between 0°C and -5°C (32°F to 23°F) for the first week of the project and the slabs were never in danger of freezing. The insulation blanket was removed in freezing temperatures, which allowed the temperature differential to exceed 19°C (35°F), which is not recommended through ACI. This caused cracking on the surface about three hours after removing the insulation.

Table 13. Compressive strength and maturity index data from cylinders cured 24 hours in the field.

Curing time (hr)	Compressive Strength, MPa (psi)	Flexural Strength, MPa (psi)	Maturity Index, °C-hr
24	15.9 (2,310)	3.25 (470)	583
30	17.4 (2,520)	3.40 (490)	735
45	18.8 (2,730)	3.57 (520)	1154
54	20.6 (2,990)	3.80 (550)	1409
72	22.0 (3,190)	3.90 (570)	1912

3.2.8 Fort Wainwright, AK

Five test slabs were created at Fort Wainwright, Alaska, in March, 2008 (Barna et al. 2010). The purpose of this project was to demonstrate CWAS technology in a full-scale trial. The goals were to be workable, finishable, have freeze protection to -5°C (23°F), develop early-age strength at or better than normal concrete, and meet acceptable air entrainment requirements. The project was completed in March 2008 at Fort Wainwright in Alaska. The test area was 7.6 m wide by 23 m long by 15 cm deep (25 ft by 75 ft by 6 in.) divided into five sections with areas of 7.6 m by 4.6 m² (25 ft by 15 ft). Each slab tested different antifreeze admixture doses.

In preparing the site, the accumulated snow at 0.5 m (18 in.) of unsuitable soil was removed. A heated tent was placed over the test area during placement of well-draining fill base material in order to ensure the desired density was achieved. The enclosure was then removed and the base material was left unprotected. Sections 1, 3, and 5 were placed first, with sections 2 and 4 placed the following day. Sensors were placed for each slab to read temperatures during and after placement.

The goals for the concrete were 28-day compressive strength of 28 MPa (4,000 psi) and 5–7% entrained air. Actual properties of each slab are shown in Table 14. Cylinders were made from each mixture, lab-cured, and each specimen was tested at ages of 1, 2, 3, and 28 days, and field-cured specimens were tested at 1, 2, 3, and 7 days after placement. All mixtures cured at room temperature achieved the target strength of 28 MPa (4,000 psi) within 48 hours, except for Mix 5, which required 11 days. All five mixtures cured in the field reached 28 MPa (4,000 psi) within five days. All mixtures reached in excess of 34 MPa (5,000 psi) at the 28 day mark.

Table 14. Plastic properties for each slab of Fort Wainwright, AK, project.

Mix	Placement Location	Temperature, °C (°F)	Slump, mm (in.)	Air Content, %
Mix 1	Section 1	16 (60)	229 (9)	5.8
Mix 2	Section 3	19 (66)	152 (6)	5.4
Mix 3	Section 5	10 (50)	229 (9)	5.7
Mix 4	Section 2	12 (53)	216 (8.5)	5.2
Mix 5	Section 4	11 (52)	229 (9)	5.5
Target Value		4-10 (40 to 50)	127-203 (5 to 8)	5 to 7

3.2.9 Fort Drum, New York

Roller-compacted concrete (RCC) is used as a means of rapid construction for massive concrete structures, typically roadways, hardstands, and dams, and has been used for several decades. Historically, RCC is completed during the typical construction season at temperatures above 7°C (45°F) and not in cold conditions. RCC is a cost effective means of concrete placement because it does not require forms or reinforcement. The goal of this project was to combine RCC technology with antifreeze concrete in order to construct a hardstand made of Cold-RCC (Cortez et al. 2010). This combination will allow larger concrete placements during cold environments.

Ten different laboratory mixtures were investigated to determine the optimal mixture proportions and admixture dosage. Laboratory testing proved an unconfined compressive strength of 25 MPa (3,600 psi) at 7 days at a constant temperature of -5°C (23°F). The field site, at Fort Drum, New York, was prepared prior to the demonstration in March of 2009 by excavating to a depth of 36 cm (14 in.) and adding a 10 cm (4 in.) layer of crushed stone atop the sand subgrade. The test site was instrumented with thermocouples in order to read and record the temperatures of the concrete. The hardstand was constructed with two continuous 3 m (10 ft) wide lanes. Four passes were used by the roller to attain the maximum density. Tearing, voids, and gouging were noticed during paving of the first lane; water content was adjusted after the first lane to improve surface appearance. On the eastern end of the project, both lanes were cured with curing compound and covered by a plastic sheet. The western end of the project was covered with polyethylene sheathing.

Aggregate properties, moisture content, placement densities, and unit weight were measured for quality control. Seven cores were taken from the test site to verify the consistency and thickness of the slab, but no mechanical properties were measured. The appearance of the cores was typical of normal RCC pavement. Within eight hours of placement a portion of the test section was opened to limited traffic. The site was open with unrestricted access after 24 hours. Overall, the mixtures showed adequate strength gain at temperatures as low as -10°C (14°F). This project was successful, but also proved that further experimentation and testing must be completed on cold-RCC as this was a much smaller scale than typically used for RCC projects.

3.2.10 Airfield damage repair

CWAS technology was applied to an existing two part, rapid setting, concrete material used for crater repair. (Oren et al. 2014). Laboratory testing was completed on the two products already in existence. The first part of the two part mixture consists of a flowable fill product which is used to rapidly fill the bulk of the crater. The second portion is a rapidly setting concrete meant to fill the final area of the crater and allow heavy-vehicle traffic. In combination with cold weather admixture systems, this development allows rapid fill and traffic to a crater that requires repair in sub-freezing temperatures. While previous test sites have focused on creating a durable product, expedient repairs were the priority in this case.

The design strength for the rapid set mixture was set at 17 MPa (2,500 psi) and 1.7 MPa (250 psi) for the utility fill tested at two hours. Research leading up to this product concluded that calcium chloride in combination with water reducer, high-range water reducer, and an accelerant was the optimal combination for reaching the desired, lowered freezing point, set time, workability, and strength. Several off-the-shelf admixtures were available and have been tested in combination to create cold weather concrete. The best performing admixture, in combination with the rapid set mixture, turned out to be aluminum sulfate, even at temperatures below -5°C (23°F). The laboratory tests showed that the best additive for the fill mixture was hot water as the mixing water and no special antifreeze admixtures.

4 Follow-on Field Site Investigations

4.1 Equipment and test descriptions

For the durability investigation, a visual inspection, nondestructive and destructive testing were completed at most field sites. Nondestructive testing enables measurement of various electrical and mechanical properties of concrete without taking samples to test in the laboratory. There are various methods used for different types of concrete and structures; the three chosen were ultrasonic pulse velocity (UPV), lightweight deflectometer (LWD), and ground penetrating radar (GPR).

4.1.1 Ultrasonic pulse velocity (UPV)

This process determines propagation velocity of longitudinal stress wave pulses through concrete. The Proceq Pundit PL-200 Ultrasonic Pulse Velocity meter was used to give information on cracks and other defects, calculate pulse velocity, surface velocity, and estimate crack depth. Pulsed longitudinal stress waves are produced by a transducer and sent through concrete. The pulses are received and converted into electrical energy by a separate receiver. The pulse velocity is calculated by dividing the distance between the transducer and receiver by the time it took to transmit the waves. After the desired area is chosen, two points approximately 15 cm (6 in.) away from each other are used to estimate pulse velocity. A coupling gel is used at each 15 cm (6 in.) interval between the transmitter and receiver and the surface of the concrete. For surface velocity, readings are taken at 15, 30, 45, and 60 cm (6, 12, 18, and 24 in.). The transmitter stayed in the “o” location while the receiver moved 15 cm (6 in.) for each reading.

4.1.2 Ground penetrating radar (GPR)

A GSSI SIR 4000 handheld logger was used in combination with a 900 MHz antenna to investigate below the surface and detect where reinforcement was. The antenna is dragged across the slab at specified locations at a constant rate. This is completed several times for each slab to mark where the reinforcement was and collect subsurface imagery. GPR uses high-frequency electromagnetic waves to acquire subsurface information. This can be used to view changes in material properties, including voids and cracks.

4.1.3 Light weight deflectometer (LWD)

The Dynatest LWD was used due to its ease of mobility and ability to operate with one user. This equipment is simple to transport but cannot be used for thick pavements. All pavements in this project were 15.24 cm (6 in.) or thinner, so use of the LWD was valid. LWD determines deflections of paved and unpaved surfaces. A falling weight mass of 10–15 kg (22 – 33 lb) is dropped on a buffer system that will transmit the load pulse through a plate resting on the testing surface and the deflection is measured. For this project, all sites were measured at four drop heights of 15, 30, 45, and 60 cm (6, 12, 18, and 24 in.). At each drop height, four readings are taken.

4.1.4 Coring

The Hilti DD 110 W hand held drill was used to bore 8 cm (3 in.) cores from each site. This equipment was chosen due to user experience. Cores were stored in double bagged plastic zip bags and wrapped in bubble wrap and foam before shipping back to CRREL. Cores were taken only from the Sault Ste. Marie, MI, and Fairbanks, AK, sites, due to restrictions in place for other field locations.

4.2 Field Site Investigations

4.2.1 Sault Ste. Marie, MI

The Sault Ste. Marie, Michigan, site on the Soo Locks was the first demonstration conducted by CRREL of CWAS under field conditions. This site was placed, and later inspected, in coordination with the USACE Detroit District, Soo Area Office stationed at the Soo Locks. The six reinforced slabs on grade contained varying admixture dosages, starting with a control with no CWAS and five additional slabs with admixtures from both MB and WRG.

Slab 1 and Slab 3 had varying admixture dosages of a proprietary admixture developed by MB called EY11. Slab 2 was the control mixture which was tented and kept at approximately 20°C for the first three days after placement (Korhonen and Brook 1996). Slab 4 was mixed using the MB admixture at the time known as Pozzutec 20+ and Slabs 5 and 6 were placed using WRG admixtures, PolarSet and DP, respectively. Apart from the control, no slabs were heated, although the EY11 low dosage mixture in Slab 1 was tented. The remaining slabs were covered with a plastic sheet

for the first seven days of curing to minimize water loss, but did not have any tenting or heating over the slab (Barna et al. 2010). The proportions of each mixture are shown in Table 15.

Table 15. Sault Ste. Marie, MI, mixture proportions.

	Slab 1	Slab 2	Slab 3	Slab 4	Slab 5	Slab 6
Property	EY11 Low	Control	EY11 High	Pozzutec® 20	Polarset®	DP®
Cement, kg/m ³ (lb/yd ³)	390 (658)	390 (658)	390 (658)	390 (658)	390 (658)	390 (658)
Aggregate, 19 mm (¾ in.) kg/m ³ (lb/yd ³)	1,044 (1,760)	1,044 (1,760)	1,044 (1,760)	1,044 (1,760)	1,048 (1,766)	1,045 (1,762)
Sand, kg/m ³ (lb/yd ³)	771 (1,300)	771 (1,300)	771 (1,300)	771 (1,300)	802 (1,352)	810 (1,365)
w/c	0.41	0.41	0.38	0.39	0.43	0.41
Slump, mm (in.)	140 (5.5)	51 (2)	140 (5.5)	152 (6)	109 (4.3)	130 (5.1)
Concrete Temperature, °C (°F)	3 (38)	12 (54)	3 (38)	4 (40)	8 (46)	7 (45)
Air Content, %	3.2	3.2	4.7	3.4	5.3	8
Avg. Pulse Velocity, m/s (ft/s)	4,652 (14,966)	3,775 (12,385)	3,259 (10,693)	3,593 (11,787)	2,635 (8,646)	2,312 (7,586)
Avg. Surface Velocity, m/s (ft/s)	4,208 (13,807)	3,346 (10,978)	2,473 (8,113)	3,256 (10,681)	2,253 (7,393)	2,208 (7,243)
Surface Modulus, MPa (ksi)	951 (138)	572 (83)	517 (75)	345 (50)	772 (112)	200 (29)
Core Strength, MPa (psi)	51 (7,350)	47 (6,770)	53 (7,720)	54 (7,840)	–	46 (6,670)

A visual inspection of the Sault Ste. Marie, MI, site showed little to no deterioration 24 years after placement. The site has been used as a vehicle pathway along the river, and a temporary mooring location for ships when needed. The slabs varied in curing processes and admixture content and dosages, but all remained resilient through years of freezing and thawing in the upper peninsula of Michigan. Of the six slabs, none exhibited excessive cracking or showed signs of deterioration. Figure 1 and Figure 2 show Slab 2 and 3, respectively.

Figure 1. Sault Ste. Marie, MI Slab 2 (control).

Figure 2. Sault Ste. Marie, MI Slab 3.

Following the visual inspection, GPR was used to determine the locations of the reinforcement grid, shown in Figure 3. GPR was also used to investigate subsurface conditions; data collected will be evaluated in further phases of the project. After locating reinforcement, coring locations were chosen near the edge and center of each slab, and to avoid severing rebar.

Three points from each slab were chosen to complete LWD and UPV testing. Data was recorded for each drop and the values for surface modulus of the center point of each slab are shown in Table 15. At the same locations, the pulse and surface velocity were recorded. These measurements were taken following the procedure described in the previous section. Cores of 8 cm (3 in.) diameter were bored from each slab allowed for further laboratory testing in later phases of the project.

Figure 3. A–C Nondestructive Testing Methods, GPR, LWD, and UPV, respectively.

4.2.2 Fort Wainwright, AK

The Fort Wainwright, Alaska site, which consists of five reinforced slabs on grade, was built as, and remains in service as, a military hardstand, shown in Figure 4. This site has been in service since March, 2008 on Fort Wainwright in Fairbanks, AK. The mixture proportions from each slab are shown in Table 16. Mixtures 1 through 4 varied in admixture dosages of the same components while Slab 4 (Mix 5) introduced a different viscosity-modifying admixture (Barna et al. 2010). All admixtures were used from the MB suite.

Figure 4. Fort Wainwright, AK, site in 1 July 2018.

Table 16. Fort Wainwright, AK, mixture proportions.

	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5
Property	Slab 1	Slab 3	Slab 5	Slab 2	Slab 4
Cement, kg/m ³ (lb/yd ³)	390 (658)	390 (658)	390 (658)	390 (658)	390 (658)
Aggregate, 19 mm (¾ in.) kg/m ³ (lb/yd ³)	1,054 (1,777)	1,045 (1,762)	1,054 (1,776)	1,048 (1,766)	1,045 (1,762)
Sand, kg/m ³ (lb/yd ³)	821 (1,383)	801 (1,350)	821 (1,383)	802 (1,352)	810 (1,365)
Glenium® 3000NS, mL/kg (fl oz/cwt)	5.2 (8.0)	4.6 (7.0)	3.2 (4.9)	3.9 (6.0)	–
Rheomac® VMA, mL/kg (fl oz/cwt)	–	–	–	–	2.6 (4.0)
Pozzutec® 20+, mL/kg (fl oz/cwt)	44.2 (68.0)	29.3 (45.0)	14.3 (22.0)	22.1 (34.0)	22.2 (34.1)
Rheocrete® CNI, L/m ³ (gal/yd ³)	20 (4.0)	15 (3.0)	7.5 (1.5)	11.5 (2.3)	11.5 (2.3)
w/c	0.39	0.39	0.38	0.38	0.47
Slump, mm (in.)	229 (9)	152 (6)	229 (9)	216 (8.5)	229 (9)
Concrete Temp., °C (°F)	16 (60)	19 (66)	10 (50)	12 (53)	(11) 52
Air Content, %	5.8	5.4	5.7	5.2	5.5
Avg. Pulse Velocity, m/s (ft/s)	4,094 (13,431)	4,368 (14,330)	3,278 (10,754)	3,477 (11,406)	3,422 (11,228)
Avg. Surface Velocity, m/s (ft/s)	3,956 (12,980)	3,920 (12,860)	3,608 (11,837)	2,854 (9,363)	3,948 (12,953)
Surface Modulus, MPa (ksi)	558 (81)	1,124 (163)	800 (116)	683 (99)	552 (80)
28-Day Strength, MPa (psi)	48 (6,945)	52 (7,518)	51 (7,338)	38 (5,508)	51 (7,335)

Initial visual inspection showed that Slab 1 and Slab 3 exhibited surface cracking, shown in Figure 5. These slabs were the first two placed of the five and contained the highest dosage of accelerator in the concrete mixture. These factors could be the reason that these two slabs incurred excessive cracking while the other three remained free of cracks.

Figure 5. Fort Wainwright, AK, Slab 3.

Nondestructive testing was completed following the same procedure for GPR, LWD, and UPV as done at the Sault Ste. Marie, MI site. Data was recorded for each LWD drop and the values for surface modulus of the center point of each slab are shown in Table 16. At the same locations, the pulse and surface velocity were recorded.

After nondestructive testing was complete for each slab, 8cm (3 in.) cores were taken from specified areas. These cores were then labeled accordingly and sealed in two plastic bags and shipped to CRREL for further laboratory testing to be completed in further phases of the project. The holes left from coring were patched with a standard bag concrete mix and cured using a plastic tarp over the area for 24–48 hours.

4.2.3 Fairbanks, AK

In addition to the Fort Wainwright site in Fairbanks, AK, two slabs were placed at the Cold Climate Housing Research Center (CCHRC). A smaller mixture, Slab 1, was used as a walkway area for a 10cm (4 in.) thick slab. Slab 2 was designed as a foundation for a future building. The mixture proportions for Mix 1 and Mix 2 are shown in Table 17.

Table 17. CCHRC mixture proportions.

Property	Slab 1 Small	Slab 2 Large
Cement, kg/m ³ (lb/yd ³)	362 (611)	362 (611)
Aggregate 19 mm (¾ in.), kg/m ³ (lb/yd ³)	1,076 (1,814)	1,076 (1,814)
Sand, kg/m ³ (lb/yd ³)	766 (1,291)	766 (1,291)
Micro Air®, mL/kg (fl oz/cwt)	0.3 (0.5)	0.3 (0.5)
Polyheed® 997, mL/kg (fl oz/cwt)	7.8 (12)	7.8 (12)
Rheocrete® CNI, L/m ³ (gal/yd ³)	25 (5)	25 (5)
Pozzutec® 20+, mL/kg (fl oz/cwt)	58.5 (90)	58.5 (90)
w/c	0.42	0.43
Slump, mm (in.)	178 (7)	178 (7)
Concrete Temperature, °C (°F)	11 (51)	9 (48)
Air Content, %	8	12
Avg. Pulse Velocity, m/s (ft/s)	3,197 (10,490)	–
Avg. Surface Velocity, m/s (ft/s)	3,524 (11,560)	–
Surface Modulus, MPa (ksi)	841 (122)	–

At the time of inspection, the building addition had been in place for several years, making a visual inspection, let alone nondestructive testing or collection cores impossible for Slab 2. For this reason, the focus of the durability inspection was placed on the smaller slab being used as a walkway. This slab showed no signs of damage and proved resilient after eleven years of use and freeze-thaw cycles. Figure 6 shows Slab 1 in its current condition.

Figure 6. CCHRC Slab 1 July 2018.

4.2.4 West Lebanon, NH

The Trues Brook Bridge project was placed with CWAS concrete on the west side of the bridge and conventional winter concrete on the east side. This site, placed in December of 2002 has gone through 15+ years of freeze-thaw cycles in the Upper Valley of New Hampshire including deicing salts and typical roadway curbing damage. The mixture design for the CWAS concrete is shown in Table 18.

Table 18. West Lebanon, NH, CWAS concrete mixture proportions.

	Mix 1
Property	Small
Cement, kg/m ³ (lb/yd ³)	513 (864)
Aggregate 19 mm (¾ in.), kg/m ³ (lb/yd ³)	1,411 (2,379)
Sand, kg/m ³ (lb/yd ³)	1,048 (1,766)
MB-VR®, mL/kg (fl oz/cwt)	0.2 (0.3)
Rheocrete® CNI, L/m ³ (gal/yd ³)	30 (6)
Polyheed® 997, mL/100kg (fl oz/cwt)	7.8 (12)
Pozzutec® 20+, mL/kg (fl oz/cwt)	58.5 (90)
w/c	0.42
Slump, mm (in.)	178 (7)
Concrete Temperature, °C (°F)	11 (51)
Air Content, %	8
Avg. Pulse Velocity, m/s (ft/s)	2,396 (7,860)

Initial visual inspection shows a clear difference between the CWAS and conventional concrete curbs. While both sides of the bridge showed cracks, the conventional concrete side of the bridge shows excessive spalling and more deterioration than the CWAS side. Figures 7–10 show a comparison between the west and east sides of the bridge after nine and fifteen years in service. The only nondestructive test conducted at this site, due to size constraints, was ultrasonic pulse velocity.

Figure 7. West Lebanon, NH, control concrete.

Figure 8. West Lebanon, NH, CWAS concrete.

Figure 9. West Lebanon, NH, control concrete.

Figure 10. West Lebanon, NH, CWAS concrete.

4.2.5 Treat Island, Maine

USACE manages three acres on the northwest side of Treat Island, Maine, a natural weather exposure site, where concrete specimens can be placed for testing durability. Factors like the weather, tidal change average of 6.7 m (22 ft), and the salt water make this site ideal for extreme weathering exposure for concrete (Figure 11). For this project, two mixtures, control and CWAS, were constructed and brought to the island. The mixture proportions and plastic properties of each are shown in Table 19. Both 10 cm by 20 cm (4 in. by 8 in.) cylinders and 8 cm by 8 cm by 29 cm (3 in. by 3 in. by 11.25 in.) beams were made and brought to the site. The beams will be measured for length change and ultrasonic pulse velocity, while the cylinders will be used for compressive strength in the future.

Figure 11. Treat Island, ME, dock.

Table 19. Treat Island, ME, mixture proportions.

Property	Admixture Amended	Control
Cement, kg/m ³ (lb/yd ³)	390 (658)	390 (658)
Aggregate 19 mm (¾ in.), kg/m ³ (lb/yd ³)	1,045 (1,762)	1,045 (1,762)
Sand, kg/m ³ (lb/yd ³)	810 (1,365)	810 (1,365)
MasterGlenium® 7620, mL/kg (fl oz/cwt)	4 (6)	4 (6)
MasterSet® FP 20, mL/kg (fl oz/cwt)	22 (34)	--
Rheocrete® CNI, mL/kg (fl oz/cwt)	29 (45)	--
MasterAir® AE200, mL/kg (fl oz/cwt)	--	0.08 (0.125)
w/c	0.39	0.39
Slump, mm (in.)	222 (9)	76 (3)
Concrete Temperature, °C (°F)	7 (44)	26 (79)
Air Content, %	3	7

4.3 Discussion of results

4.3.1 Sault Ste. Marie, Michigan

The Sault Ste. Marie, Michigan, site is a useful addition to the durability investigation due to the inclusion of a control mixture. For this phase of the investigation, nondestructive testing was the main focus for analysis and a further study will include experimentation on the cores taken from each site. Cores were taken from the site in both 1994 and 2018. In 1994, strengths were taken from each showing that all slabs met or exceeded the control concrete. The surface modulus estimated using the LWD showed that four of the five admixture amended slabs were at least 75% of the control value after 24 years in service. The slab with the lowest performing surface modulus was Slab 6. When inspecting surface and pulse velocity data, all slabs except Slab 5 and Slab 6 also follow the trend of being at least 75% of the control value for the same test. The commonality of the two slabs is that they are both products of the same WRG admixture distributor. Since no slabs showed significant cracking or deterioration, further testing on the cores will have to be completed to understand the long-term durability of all of these concrete mixtures.

4.3.2 Fort Wainwright, Alaska

The next site investigated was at the Fort Wainwright Army Post in Fairbanks, Alaska. Unfortunately, this site was not placed alongside a

control to provide simple evaluation. When comparing LWD data of the surface modulus of each slab, Slab 2 outperformed the others by at least 40%. This slab also showed no signs of excessive cracking or deterioration during the visual inspection. Following behind with second greatest surface modulus was Slab 3, which showed signs of surface cracking. UPV data showed again that Slab 2 had the highest pulse velocity and second highest surface velocity, behind Slab 1. This data is not the determining factor for long-term durability, but will be used in combination with future experiments to determine the optimal testing plan for long-term durability.

4.3.3 Fairbanks, Alaska

The second site in Fairbanks, Alaska, at the CCHRC only had one slab available for inspection, making it difficult to compare to a similar mixture. The admixtures and dosages used did not closely resemble those of Fort Wainwright, Sault Ste. Marie, or West Lebanon. However, the values recorded from UPV fell in a range between those from Sault Ste. Marie and Fort Wainwright. The same is true for the surface modulus data of the center of the slab collected by the LWD. Similarly to previous sites, further experimentation with the cores collected from the CCHRC site will provide a more holistic view of the concrete.

4.3.4 West Lebanon, New Hampshire

The Trues Brook Bridge site in West Lebanon, New Hampshire, has been through 15+ years of harsh New Hampshire winters including frequent freeze thaw cycles and exposure to deicing salts. Figures 7–10 show the difference between the control and CWAS concrete placed at the same time. The figures show cracking on each section, but a significant amount of scaling on the control side of the bridge. This examination encouraged the introduction of scaling tests for the next phase of this project. Unfortunately, nondestructive test data other than a visual inspection has not been collected in the past, but will be collected annually in the future.

5 Conclusions and Recommendations

5.1 Conclusions

The primary findings of this project suggest that concrete placed in below freezing weather using CWAS technology is durable with regard to freezing and thawing cycles, including exposure to deicing chemicals. These findings are based primarily upon the assessment of multiple field sites that range from one to two decades in age. In many cases, CWAS concrete exhibits superior durability when compared to conventional cold weather concrete placed simultaneously and exposed to the same conditions. Past technical reports have documented the reduced cost and improved constructability of CWAS when compared to conventional sub-freezing concrete placement techniques.

Despite the cost and construction advantages of CWAS reviewed in this report, including preliminary findings that indicate highly durable concrete made with these techniques, use of this technology in the last decade has been minimal. The authors of this report have identified two critical needs that must be addressed to further the ultimate utility and application of CWAS, which are summarized as follows:

1. The mechanisms responsible for cold-climate durability in CWAS concrete have not been identified; this makes prediction of in-service CWAS longevity challenging.
2. No published specifications, tools, or guidance documents exist to help implement CWAS technology broadly across civil and military applications.

These needs will be addressed through a comprehensive laboratory testing program to be implemented in further stages of the project. An outline of this testing program is described in section 5.3

5.2 Field specimen analysis

Two hypotheses have been formulated to guide the analysis of the field specimens collected as part of this work:

1. The use of cold weather admixture systems during placement of concrete in sub-freezing conditions provides resistance to damage from freeze-thaw beyond what is provided by a proper air-void system.
2. The early-age freezing-point depression provided by cold weather admixture systems persists with time, effectively reducing the number of freeze-thaw cycles experienced by the concrete.

Testing these hypotheses requires characterization of the air-void system within the hardened specimens using hardened air void analysis via ASTM 2016d, determination of the chemical composition of the water within the specimens, and measurement of the temperature at which ice begins to form within the specimens.

In past experiments involving CWAS, the freezing point depression of the fresh concrete has been identified by placing small samples of fresh concrete with embedded thermocouples in a beverage cooler containing dry-ice. The temperature of the concrete was measured, and the freezing point was identified as a level spot in the cooling curve caused by the phase change of water to ice (Korhonen et al. 2004). While this test is expedient for fast measurements of fresh concrete in the field, the level spot in the cooling curve rapidly becomes harder to discern over the first 30 hours of specimen aging (Korhonen et al. 1998). For this reason, a more sensitive thermal analysis technique will likely be required to measure the freezing temperature of the pore solution within mature, hardened concrete specimens.

A potential technique under consideration for measurement of the pore solution freezing point in specimens taken from field sites is differential thermal analysis (DTA). DTA is a classical analysis technique that is useful for detecting phase changes in materials (Vold 1949), and involves comparing the temperatures of a sample as well as a thermally inert, thermal-mass equivalent reference. The reference must be composed of a material which will not undergo any phase change or thermal reaction over the range of temperatures being investigated. Both the sample and reference are exposed to the same rate of heating and cooling, and the differential temperature, including the reference temperature, are measured. A phase change in the sample is obvious when the differential temperature spikes (Smykatz-Kloss 2012). An exact procedure for applying this test to measurement of hardened concrete freezing will be developed early in future stages of the project.

Pore solution analysis via expression (Barneyback and Diamond 1981), and possibly via in situ leaching (Li et al. 1999) will be employed to ascertain potential differences in pore solution chemistry that may stem from the high dosages of accelerating/antifreeze admixture used in field sites. This information will be useful to determine the presence of any CWAS admixture chemicals that remain in the concrete. Some precedent exists for the detection of calcium nitrite in concrete pore solution at ages greater than 100 days. Li et al. (1999) found that the concentration of nitrite in pore solution (via in situ leaching) of concrete and mortar made with corrosion-inhibiting admixtures increased rapidly at early ages (5–20 days) and more slowly at later ages (20–102 days). The rate of increase was still positive at the last measurement.

The final component of the field specimen analysis will be compilation of climate data local to each field location to estimate the freeze thaw history of collected specimens. This information should help to correlate the number of actual freeze thaw cycles experienced by CWAS concrete to measurements of freeze thaw resistance per ASTM C666 standard testing, and provide comparative metrics for sites of different ages.

5.3 Laboratory testing

The ultimate goal of the laboratory testing program is to understand how CWAS concrete achieves durable performance in the field. This includes assessing the relative effects of the air void system as well as the contribution of the CWAS themselves. CWAS concrete with varying air content will be created and tested to evaluate this. If the freezing point of CWAS concrete is lower than that of conventional concrete, the number of freeze-thaw cycles experienced will be lower; adjustments to the durability thresholds defined in ASTM C666 will be developed to compensate. CWAS concrete in the field appears to exhibit superior resistance to scaling when compared to conventional cold weather concrete, however, this has never been tested in the laboratory. Finally, the feasibility of casting CWAS concrete incorporating pozzolanic materials will be evaluated. Most state agencies require pozzolans for any new construction. To date, no attempts have been made at CRREL to combine CWAS with supplementary cementitious materials.

The laboratory program will include common tests for plastic properties including slump, volumetric air, setting time and unit weight, and standard hardened properties such as compressive strength and elastic

modulus. The DTA freezing point test developed for use with field specimens will be implemented to measure the change in freezing point of CWAS with age. Specimens will also be created to measure the resistance of CWAS concrete to scaling from deicing chemicals (ASTM 2012a). Freeze thaw testing will be implemented using a modified procedure based on ASTM C666; modifications will be informed based on the freezing point of the hardened CWAS concrete, including the freeze-thaw history of the specimens in the field. The laboratory testing program may also include measurements of concrete permeability, either by surface resistivity (AASHTO TP 358 2017), or rapid chloride ion permeability (ASTM 2012b). The permeability of concrete plays an important role in freeze thaw durability by controlling the rate at which the microstructure can become saturated.

5.4 Guidance and tools for implementation of CWAS

Future plans for this project will work toward the development of formalized guidance for the use of CWAS by both civil and military agencies. Practical use of CWAS faces the following challenges:

1. Sensitivity of antifreeze/accelerating admixtures to initial and ambient temperature resulting in either frozen concrete or excessively short working times;
2. Staging requirements associated with compensating for the high water content of calcium nitrite and nitrate admixtures;
3. Variable compositions of commercially available admixture formulations;
4. Variations in mixture proportions including water to cement ratio and cement content;
5. Variability in the self-heating potential of concrete elements with changing minimum dimension.

The most useful deliverable for practical implementation of CWAS would be an admixture dosage calculator, which can take the variables outlined above into account. A key step in development of this calculator is the determination of the precise chemical composition of existing accelerating/antifreeze admixture formulations. This will be accomplished with wet chemical analysis using CRREL personnel and facilities; the information obtained will be applied to inform a standardized dosage approach using common admixture chemicals purchased in bulk. The possibility of dosing admixtures in powdered form will also be

investigated, with the potential to reduce the staging complications associated with high admixture water content.

In regions where temperature rise in mass concrete is a concern, thermal finite different modelling is commonly employed to predict temperature rise based on placement temperature, ambient temperature, and a calculated adiabatic temperature rise (ATR) for the portland cement content of the mix (ACI 2005). A simple, one dimensional model can be created to model concrete temperature over time using this method, which could be applied to mixtures containing CWAS admixtures. To support this, an experimental apparatus will be developed to cure concrete cylinders under precisely controlled dynamic thermal conditions. This will enable both adiabatic temperature rise measurement through match curing as well as curing of cylinders as though they were part of a much larger structure, and will provide data points to quantify the effects of varying cement content and minimum dimension on the temperature rise associated with a range of CWAS dosages.

The chemical and thermal data collected to characterize CWAS performance with dosage will be supplemented with standard plastic and hardened concrete tests as described in the laboratory testing subsection in the concrete durability testing plan. This data will also be combined with historical data from CRREL. The resulting database will be used to train a machine learning model, which will ultimately form the underpinnings of a simple dosage calculator, possibly in the form of a smartphone application. This calculator may be transformed into a nomograph for expedient usage in adverse conditions. If possible within the time constraints of the project, the functionality of the developed calculator will be evaluated in field trials.

References

- AASHTO. 2017. *Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration*. AASHTO TP 358. Washington, DC: American Association of State and Highway Transportation Officials (AASHTO).
- ACI. 2005. *Guide to Mass Concrete*. ACI 207.1 R-05. Farmington Hills, MI: American Concrete Institute (ACI).
- ACI. 2016. *Concrete Terminology*. ACI CT-13. Farmington Hills, MI: American Concrete Institute (ACI).
- Archie, G. E. 1942. The electrical resistivity log as an aid in determining some reservoir characteristics. In *Transactions of the AIME* 146(01):54–62.
<https://doi.org/10.2118/942054-G>.
- ASTM. 2018. *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. ASTM C39/C39M-18. West Conshohocken, PA: ASTM International.
www.astm.org.
- ASTM. 2017. *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method*. ASTM C231/C231M-17. West Conshohocken, PA: ASTM International. www.astm.org.
- ASTM. 2016a. *Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method*. ASTM C173. West Conshohocken, PA: ASTM International.
www.astm.org.
- ASTM. 2016b. *Standard Test Method for Determining the Apparent Chloride Diffusion Coefficient of Cementitious Mixtures by Bulk Diffusion*. ASTM C1556. West Conshohocken, PA. www.astm.org.
- ASTM. 2016c. *Standard Specification for Cold-Weather Admixture Systems*. ASTM C1622/C1622M-10. West Conshohocken, PA. www.astm.org.
- ASTM. 2016d. *Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*. ASTM C457. West Conshohocken, PA: ASTM International. www.astm.org.
- ASTM. 2015. *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*. ASTM C666. West Conshohocken, PA. www.astm.org.
- ASTM. 2012a. *Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals*. ASTM C672/C672M-12. West Conshohocken, PA: ASTM International. www.astm.org.
- ASTM. 2012b. *Standard Test Method for Electrical Indication of Concretes Ability to Resist Chloride Ion Penetration*. ASTM C1202. West Conshohocken, PA: ASTM International. <http://www.astm.org/Standards/C1202.htm>.
- ASTM. 2012c. *Standard Test Method for Determining the Penetration of Chloride Ion into Concrete by Ponding*. ASTM C1543. West Conshohocken, PA. www.astm.org.

- ASTM. 2012d. *Standard Test Method for Bulk Electrical Conductivity of Hardened Concrete*. ASTM C1760. West Conshohocken, PA. www.astm.org.
- Barna, L. A, P. M. Seman, and C. J. Korhonen. 2010. *Cold Weather Admixture Systems Demonstration at Fort Wainwright, Alaska*. TR-10-6. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Barneyback, R. S., and S. Diamond. 1981. Expression and analysis of pore fluids from hardened cement pastes and mortars. *Cement and Concrete Research* 11(2):279–285. [https://doi.org/10.1016/0008-8846\(81\)90069-7](https://doi.org/10.1016/0008-8846(81)90069-7)
- Berke, N. S., and M. C. Hicks. 2004. Predicting long-term durability of steel reinforced concrete with calcium nitrite corrosion inhibitor. *Cement and Concrete Composites* 26(3):191–198. [https://doi.org/10.1016/S0958-9465\(03\)00038-6](https://doi.org/10.1016/S0958-9465(03)00038-6)
- Cortez, E., L. Barna, C. Smith, and T. Peltz. 2010. *Cold Roller-Compacted Concrete for Roads and Hardstands on Army Installations in Cold Regions*. TR-10-08. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC)..
- Department of Defense (DoD). 2016. *Report to Congress on Strategy to Protect United States National Security Interests in the Arctic Region*. Washington DC: Office of the Undersecretary of Defense (OUSD) (Policy). Department of Defense.
- Dong, S., E. Callagon La Plante, X. Chen, M. Torabzadegan, M. Balonis, M. Bauchy, and G. Sant. 2018. Steel corrosion inhibition by calcium nitrate in halide-enriched completion fluid environments. *Npj Materials Degradation* 2(1):32.
- Korhonen, C. J., B. Charest, and K. Romisch. 1997a. *Developing New Low-Temperature Admixtures for Concrete. A Field Evaluation*. SR-97-9. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Korhonen, C. J. 1990. *Antifreeze Admixtures for Cold Regions Concreting: A Literature Review*. SR-90-32. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Korhonen, C. J. 2006. *Extending the Season for Concrete Construction and Repair. Phase II-Defining Engineering Parameters*. TR-06-8. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Korhonen, C. J., and J. W. Brook. 1996. *Freezing Temperature Protection Admixture for Portland Cement Concrete*. SR 96-28. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Korhonen, C. J., E. R. Cortez, T. A. Durning, and A. A. Jeknavorian. 1997b. *Antifreeze Admixtures for Concrete*. SR-97-26. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).

- Korhonen, C. J., P. Semen, and L. Barna. 2004. *Extending the Season for Concrete Construction and Repair*. TR-04-2. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Korhonen, C. J., and P. Semen. 2005. *Placing Antifreeze Concrete at Grand Forks Air Force Base*. ERDC/CRREL TR-04-9. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Korhonen, C. J., J. Hughes, F. Best, and G. Mass. 1998. Low-temperature repair of the ice condenser floor slab at the Sequoyah Nuclear Power Plant. In *Cold Regions Impact on Civil Works*. 258–70.
- Li, L., A. A. Sagüés, and N. Poor. 1999. In situ leaching investigation of pH and nitrite concentration in concrete pore solution. *Cement and Concrete Research* 29(3):315–21. [https://doi.org/10.1016/S0008-8846\(98\)00224-5](https://doi.org/10.1016/S0008-8846(98)00224-5)
- Litvan, G. G. 1988. *The Mechanism of Frost Action in Concrete: Theory and Practical Implications*. National Research Council Canada, Institute for Research in Construction.
- Li, W., M. Pour-Ghaz, J. Castro, and J. Weiss. 2011. Water absorption and critical degree of saturation relating to freeze-thaw damage in concrete pavement joints. *Journal of Materials in Civil Engineering* 24(3):299–307.
- Monteiro, P. 2006. *Concrete: Microstructure, Properties, and Materials*. New York, NY: McGraw-Hill Publishing.
- Neville, A. M. 1995. *Properties of Concrete*. Vol. 4. London: Longman.
- Oren, J. I., R. D. Moser, V. Boddu, C. A. Weiss Jr, and J. Clausen. 2014. *Laboratory Evaluation of Expedient Low-Temperature Admixtures for Runway Craters in Cold Weather*. Hanover, NH: Cold Regions Research and Engineer Laboratory (CRREL), Engineer Research and Development Center (ERDC).
- Portland Cement Association (PCA). 1998. Control of air content in concrete. *Concrete Technology Today* 19(1):1–8. https://www.cement.org/docs/default-source/fc_concrete_technology/pl981.pdf?sfvrsn=2&sfvrsn=2
- Smykatz-Kloss, W. 2012. *Differential Thermal Analysis: Application and Results in Mineralogy*. Vol. 11. Berlin, Germany: Springer Science and Business Media.
- Snyder, K. A. 2001. The relationship between the formation factor and the diffusion coefficient of porous materials saturated with concentrated electrolytes: Theoretical and experimental considerations. *Concrete Science and Engineering* 3(12): 216–24.
- White House. 2013. *National Strategy for the Arctic Region*. Washington, D.C.: Government of the United States of America.
- Vold, Marjorie J. 1949. Differential thermal analysis. *Analytical Chemistry* 21 (6): 683–88.

REPORT DOCUMENTATION PAGE				<i>Form Approved</i> OMB No. 0704-0188	
Public reporting burden for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing this collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden to Department of Defense, Washington Headquarters Services, Directorate for Information Operations and Reports (0704-0188), 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302. Respondents should be aware that notwithstanding any other provision of law, no person shall be subject to any penalty for failing to comply with a collection of information if it does not display a currently valid OMB control number. PLEASE DO NOT RETURN YOUR FORM TO THE ABOVE ADDRESS.					
1. REPORT DATE (DD-MM-YYYY) October 2019		2. REPORT TYPE Final report		3. DATES COVERED (From - To)	
4. TITLE AND SUBTITLE Long-Term Durability of Cold Weather Concrete Phase I Report				5a. CONTRACT NUMBER	
				5b. GRANT NUMBER	
				5c. PROGRAM ELEMENT NUMBER 62784/T53	
6. AUTHOR(S) Danielle E. Kennedy, Benjamin E. Watts, Charles E. Smith, and Jared I. Oren				5d. PROJECT NUMBER	
				5e. TASK NUMBER	
				5f. WORK UNIT NUMBER	
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) U.S. Army Engineer Research and Development Center (ERDC) Cold Regions Research and Engineering Laboratory (CRREL) 72 Lyme Road Hanover, NH 03755-1290				8. PERFORMING ORGANIZATION REPORT NUMBER ERDC/CRREL TR-19-22	
9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES) Headquarters, U.S. Army Corps of Engineers Washington, DC 20314-1000				10. SPONSOR/MONITOR'S ACRONYM(S)	
				11. SPONSOR/MONITOR'S REPORT NUMBER(S)	
12. DISTRIBUTION / AVAILABILITY STATEMENT Approved for public release; distribution is unlimited.					
13. SUPPLEMENTARY NOTES					
14. ABSTRACT The placement of conventional concrete in sub-freezing temperatures necessitates costly, labor-intensive measures to prevent freezing and ensure adequate early strength. An alternative method of sub-freezing concrete construction employs antifreeze admixtures which concurrently depress the freezing point and accelerate the hydration of fresh concrete. This concrete mixture has been applied successfully in cold regions where there are sub-freezing temperatures, ultimately enabling an extension of the construction season. In the last two decades, multiple field demonstrations of this technology have been implemented, many of which remain in service. The materials and mixtures used at these sites were evaluated in the short term with laboratory testing, however, most have not been inspected over time to evaluate their long-term durability in service. This document includes characteristics of durability, including nondestructive and destructive testing measures to determine durability under long-term field conditions.					
15. SUBJECT TERMS		Concrete--Service life Concrete construction--Cold weather conditions		Concrete--Effect of temperature on Concrete--Additives	
16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES 63	19a. NAME OF RESPONSIBLE PERSON
a. REPORT UNCLASSIFIED	b. ABSTRACT UNCLASSIFIED	c. THIS PAGE UNCLASSIFIED			19b. TELEPHONE NUMBER (include area code)