

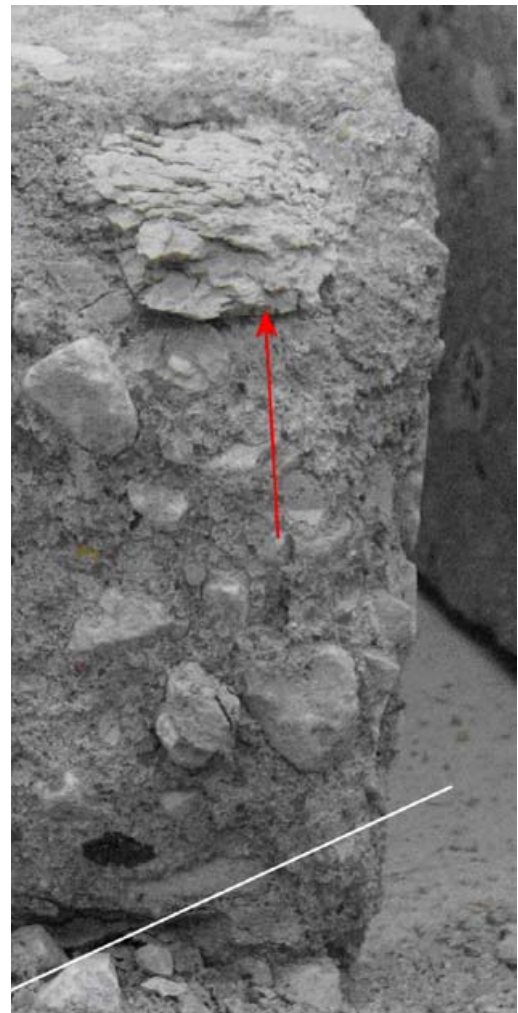
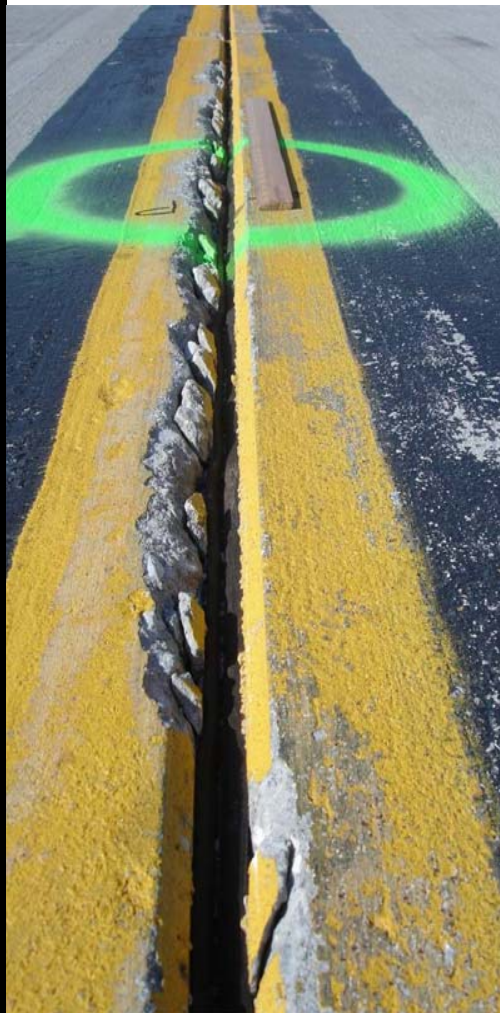


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Investigation of Sliver-Spall Damage at Offutt Air Force Base, Nebraska

Toy S. Poole and Raymond S. Rollings

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Toy S. Poole

*Geotechnical and Structures Laboratory
U.S. Army Engineer Research and Development Center
5980 Halls Ferry Road
Vicksburg, MS 39180-6199*

Raymond S. Rollings

*Rollings Consulting LLC
27 Park Square North
Beaufort, SC 29907-2159*

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Civil Engineering Support Agency
Tyndall Air Force Base, FL 32403

Abstract: This report summarizes an investigation into the cause of excessive sliver-spall damage in new airfield concrete after first winter exposure in Nebraska. The damage was particularly concentrated along joints that experienced more service loads than average; however, other sections of high-load joints showed no damage. The results showed that a combination of deficiencies in air entraining, sporadic occurrence of nondurable coarse aggregate and, possibly, early application of deicing salts contributed to the damage. The amount of new damage was notably lower after the second winter of exposure. Several recommended revisions to current concrete practice were made.

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Preface

The work described in this report was authorized by the U.S. Air Force Civil Engineering Support Agency (AFCESA), Tyndall AFB, Florida. Dr. Craig Rutland is Director of AFCESA. Dr. Toy S. Poole of the Concrete and Materials Branch (CMB), Geotechnical and Structures Laboratory (GSL), U.S. Army Engineer Research and Development Center (ERDC), was the Principal Investigator and primary author of this report. Dr. Raymond S. Rollings was Technical Monitor for the project and contributed to this report. Dr. Charles A. Weiss, CMB, interpreted the XRD data, and Linda Ragan, CMB, executed the ASTM C 457 analyses.

Toney K. Cummins, Chief, CMB, monitored the investigation at ERDC–GSL, under the general supervision of Dr. Larry N. Lynch, Chief, Engineering Systems and Materials Division; Dr. William P. Grogan, Deputy Director, GSL; and Dr. David W. Pittman, Director, GSL.

COL Gary E. Johnston was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

Unit Conversion Factors

Multiply	By	To Obtain
feet	0.3048	meters
inches	0.0254	meters
miles (U.S. statute)	1,609.347	meters
pounds (force) per square inch	6.894757	kilopascals

Summary

The purpose of the work reported in this document was to determine the cause of excessive sliver-spall damage in new airfield concrete pavement at Offutt AFB, Nebraska, and to recommend changes in practice to prevent the problem in future construction. The concrete was placed in mid- to late 2006. By spring 2007, major damage to joints was noted in some areas. The damage was particularly concentrated along joints that experienced more service loads than average, although, notably, other sections of these high-load joints totally lacked damage.

The U.S. Army Engineer Research and Development Center executed an investigation in summer 2007 and concluded that the usual sources of joint spall were not the causes of the damage at Offutt. However, the report of this investigation did not conclusively identify the cause of the problem.

As part of the work reported herein, a larger sampling and analytical investigation was executed, focusing on standard laboratory testing of concrete for resistance to cycles of freezing and thawing, properties of the entrained air, and effects of potassium acetate deicing salts. Also included was work on monitoring the pavement for additional damage during the winter of 2007-2008, review of quality assurance data on air content during construction, concrete maturity issues, interviews with concrete experts in the area near Offutt, and analysis of other concretes for comparative purposes.

Significant conclusions are summarized below.

- The concrete was not durable to repeated cycles of freezing and thawing as conducted in ASTM C 666.
- Regarding air entraining, there was apparent loss of air during placing and consolidation, and critical air void parameters did not conform to American Concrete Institute recommendations.
- Significant amounts of coarse aggregate not durable to cycles of freezing and thawing were identified.

- Relatively brief exposure of concrete to deicing salt solution affected durability of cement paste and some aggregates to cycles of freezing and thawing.
- Damage in the second winter was considerably less than in the first.
- Some combinations of air-entraining deficiencies, nondurable aggregate, and deicing salt exposure were causative factors of the damage. Mechanical service loads probably played a role only in revealing previously damaged concrete.
- Concrete placed in mid-October probably was retarded in strength gain by cold weather, but this was concluded not to have negatively impacted the concrete. Most of the concrete that suffered damage was placed at least several weeks earlier and would not have been significantly affected by cold weather.

The following recommendations are offered:

- Investigate use of the air void analyzer as a tool for more careful monitoring of in-place properties of air entraining.
- Consider revising the total air requirement to compensate for losses of air during placing and consolidation.
- Revise the current aggregate acceptance guidance.
- Conduct a more detailed investigation to determine whether the newer deicing salts create a worse durability situation than has been known for the traditionally used salts.
- Consider the use of the maturity method for tracking strength development of in-place concrete when weather is below 10 °C for significant periods.

1 Introduction

Background

Airfield concrete was placed at Offutt AFB, Nebraska, in mid- to late 2006. By spring 2007, there was major damage to joints in some areas. The damage was particularly concentrated along joints that experienced more service loads than average, but notably there were also sections of these high-load joints that totally lacked damage.

During summer 2007, the U.S. Army Engineer Research and Development Center (ERDC) executed a limited analysis of the problem. One of the suspected causes was improper air entraining in the concrete near the joints. However, the investigation concluded that, although the air entraining was somewhat less than recommended criteria, it was within limits and was thought to be only a minimal problem, at least in the early history of the concrete. No other deficiencies were noted. The report summarizing the investigation (Poole et al. 2007) concluded that the cause of the damage was mechanical in nature.

In a review, the U.S. Air Force (USAF) took issue with the conclusion that the air entraining was not a significant part of the problem. Poor air entraining had been found in a number of previous investigations of similar problems. The work reported herein was commissioned for the purpose of investigating the problem in more detail.

Spalling

Spalling is the deterioration of the portland cement concrete along pavement slab edges. This deterioration is usually confined within a few feet of the slab edge and does not extend through the full depth of the pavement; typically, cracking and deterioration of the concrete intersects the joint face. Spalling produces loose fragments of concrete that are a potential foreign object damage (FOD) hazard for aircraft. Consequently, spalling is considered a potentially serious airfield pavement defect by the USAF, as it can potentially compromise aircraft safety.

UFC 3-260-16FA, *Airfield Pavement Condition Index Survey Procedures* (Department of Defense 2004), recognizes two types of joint spalling:

rigid pavement distress no. 74 (transverse and longitudinal joint spalling) and distress no. 75 (corner spalling). Each spalling distress type is rated in three severity levels (low, medium, and high) as a function of spall length, cracks, severity of cracks, and fragments. The spall severity rating primarily reflects the potential FOD and tire-cutting hazard posed by the spall. Several examples of spalling are shown in Figure 1.



Figure 1. Examples of runway joint spalling: (top left) low-severity spalling, Elmendorf AFB; (top right) medium-severity spalling, Offutt AFB; (bottom) high-severity spalling, Bagram AB

The USAF also has an optional FOD index that is calculated from the specific pavement condition index (PCI) distresses that may generate FOD hazards, and this calculation includes both spalling distresses (Air Force Civil Engineering Support Agency (AFCESA) 2004). The FOD index is optional and is not used everywhere. However, depending on the aircraft mission, location, and similar factors, FOD potential can become one of the major criteria for determining the serviceability of an airfield pavement for USAF applications (AFCESA 2004).

Spalling is a common distress on rigid airfield pavements and requires prompt maintenance actions, because of the potential FOD hazard to aircraft. At the lowest severity level, maintenance may consist of simple sweeping or hand removal of debris to reduce FOD hazard. In more severe cases, it may require partial depth patches or even complete slab replacement or overlay.

Causes of spalling

Spalling may be caused by a wide range of phenomena including loads, poor construction, inadequate materials, and poor durability. Specific individual spalling mechanisms are discussed in the following sections, although in the field, spalling often arises from a combination of factors.

Structural loads. As pavement structural capacity is reached and exceeded by applied aircraft traffic, pavement deflections and formation of structural transverse and longitudinal cracks can lead to extensive spalling along joints, corners, and cracks. In one of the Corps of Engineers' full-scale accelerated traffic tests used to develop military rigid airfield structural design procedures, spalling at joints developed under traffic to such an extent that the section was considered failed even without any of the cracking normally used to define structural failure (Rollings and Witeczak 1990). An example of spalling caused by excessive structural loading is shown in Figure 2. In such cases, the pavement structural capacity is being exceeded, and major rehabilitation is warranted. Ad hoc patching is inadequate for such conditions.



Figure 2. Extensive spalling along joints and cracks caused by excessive structural loading, Bagram AB runway.

Mechanical loads. Loads from sources other than traffic may also cause localized failure of the pavement along the joint and may lead to spalling.

Sawing of contraction joints occurs on the day of concrete placement. Waiting longer than the day of placement for contraction joint sawing often leads to uncontrolled cracking. If the sawing is undertaken too soon

before the concrete has gained adequate strength, raveling of the saw cut and expulsion of individual aggregate particles may occur during sawing. Poor sawing techniques, dull saw blades, and similar problems can also cause these problems during sawing of the initial contraction joint or in the later sawing of the joint sealant reservoir for contraction and construction joints. Usually, such problems are noticed immediately during sawing, and the problem is rectified. Consequently, spalling from sawing operations tends to be localized and of limited extent. No USAF project of which the authors are aware had sufficient sawing damage to be of any significant concern.

Grinding uses closely spaced saw blades to cut shallow grooves in the pavement. The rotation of the saw blades snaps off the thin fin of concrete between the blades, leaving a rough-textured surface. This is a routine technique to restore surface smoothness, improve skid resistance, and remove thin surface defects on concrete pavements. In 2005, a new runway at Laughlin AFB had major smoothness problems, and the contractor elected to restore the smoothness by grinding the concrete surface. This required grinding some areas in excess of a 1/2-in. deep, and the fins between the saw blades failed to break off. These fins had to be removed by scraping with a motor grader. Extensive popouts and some spalling developed on this pavement, and the cause was the combination of the grinding and scraping operations that left elongated aggregate particles with a damaged aggregate-paste bond interface (Rollings 2006). This is the only known USAF case in which grinding caused surface damage, and this was with unusually deep grinding.

Cold milling uses carbide-tungsten teeth mounted in a helical pattern on a rotating drum to remove thin layers of concrete surface prior to overlay. This surface preparation technique often causes spalling at transverse joints, although since the pavement is normally in the process of being overlaid, such damage is usually tolerated. However, shotblasting techniques have largely displaced cold milling for surface preparation of concrete for overlays (Semen and Rollings 2005).

Snowplows are sometimes cited as a potential cause of airfield pavement spalling. However, airfield snow and ice control are routine USAF activities that are highly controlled; are overseen closely by base management systems, use standardized equipment, procedures, and chemicals, and are manned by personnel specifically trained in snow and ice removal

procedures (Department of the Air Force 1999). Hundreds of snow and ice control activities are conducted annually across the Air Force without spalling becoming a problem, and have been conducted for decades. Past USAF experience is that concrete pavement in good condition will not be damaged by routine snowplowing and ice control activities following USAF guidance. There may be isolated scrapes and chips, any areas not meeting specified smoothness tolerances may be damaged, raised obstructions such as lights are vulnerable to scraping, and unsound concrete may be damaged. However, the USAF has no reported damage from its routine snow and ice control equipment operations—as would be the case if routine snow and ice control operations were causing spalling and generating FOD hazards.

Improper equipment usage can, however, cause damage. Myrtle Beach and Cannon AFB have both had past damage to runway centerlines from improper use of equipment by inexperienced personnel, but these are the only major snowplow-caused spalling incidents that required investigation by AFCESA (Rollings 1998, Rollings et al. 2008). Some examples of snowplow damage to USAF pavements are shown in Figure 3.



Figure 3. Examples of snowplow damage to USAF airfield pavements: (top left) scraping around raised obstacle; (top right) inexperienced operator damaged centerline, compounded by improper grooving pattern passing through crowned centerline; (bottom) pavement grade change outside of smoothness tolerances resulting in dragging of blade tip of snow-clearing equipment.

Construction placement and finishing operations. Poor construction practices were observed to be the largest single contributor to early-age spalling problems in past USAF studies (Rollings 1998; Rollings et al. 2008). These poor practices are forbidden in military pavement specifications, but enforcement of such requirements varies between projects. Government inspection, contractor quality control, and construction crew skill and education are crucial to preventing these problems in the field.

Airfield pavement concrete is produced at a stiff consistency to obtain good strength and durability, and is intended to be placed by machine with minimal hand labor to finish. The desired surface on airfield concrete should provide just sufficient mortar (~1/8 in.) on the surface to allow texturing. When extensive hand finishing manipulates the surface or when water is added to the concrete surface to aid in finishing, it often builds up a thick layer of mortar that has a high water-cement ratio and low entrained air with corresponding poor strength and durability. These overworked areas are often along longitudinal edges and tend to be localized. The concrete below the mortar-rich surface or elsewhere on the slab is usually fine. This weaker and less durable surface mortar is now prone to spalling and scaling from environmental factors or applied loads. The need to use extensive hand finishing on airfield concrete pavements usually results from poor concrete mixture properties, inadequate placement equipment, a misguided attempt by the crew to get a “better-looking” surface, or some combination of such factors.

The equipment used to place military airfield-quality concrete must be able to handle, consolidate, and finish the stiff 3/4- to 3-in. slump concrete material in a single pass. If the equipment is inadequate for the task, intense hand finishing, addition of water, or repeated passes of the equipment are typically needed to overcome the equipment inadequacies. This often results in a layer of mortar on the surface or along joints that is of poor durability. Lightweight bridge-deck finishers, various rotating tube screeds, tube floats, and similar equipment have often proven inadequate for the placement of stiff airfield concrete mixtures. Such equipment is fine for other applications but often encounters difficulties placing the thick sections of stiff concrete typical of USAF airfield pavements.

When a slip-form paver pulls forward, the unsupported edge may slump downward, and attempts to repair this by hand with mortar are almost always unsatisfactory (Figure 4). These mortar repairs to the downward-slumped concrete surface often result in severe spalling that can be continuous down whole lanes (Rollings 1998). If edge slump occurs on a slip-formed pavement, poor concrete mixture proportions or varying proportions are often the cause of the problem. Edge slump cannot be repaired, so the paving process should be stopped and adjustments made in the concrete mixture proportions or equipment to resolve the issue.



Figure 4. Examples of edge-slump problems with slip-form paving at Travis AFB: (top left) example of edge slump; (top right) attempts to repair by hand; (bottom) failure of edge-slump repairs.

Curing. Proper curing is crucial for portland cement concrete's durability and strength. Poor curing will make the concrete more vulnerable to damage from environmental effects such as freezing and thawing (i.e., poor curing results in higher permeability, allowing more ready saturation) or from mechanical or traffic loading.

Elmendorf AFB developed an unusual case of spalling, the root cause of which lay in poor curing of the concrete joints on a runway rehabilitation (Rollings et al. 1998). When transverse joints were initially cut, the sawing operation disturbed the curing membrane in the area of the cut joints, and the curing membrane was not re-established as required in the specifications. This runway was reopened to traffic on an expedited schedule, and high-tire pressure F-15E aircraft caused minor cracking along the joint walls that stabilized as the concrete continued to gain strength. With the onset of freezing weather some months later, moisture in the pre-existing

cracks led almost all of the transverse joint in trafficked areas to develop spalling over a 2-day period. An in-depth laboratory, analytical, and field assessment of this spalling problem (Rollings et al. 1998) found only this scenario of events centered on initial curing difficulties could adequately explain the observed phenomena.

At Wright-Patterson AFB, curing membrane was applied before the bleed-water had evaporated. This bleed-water was trapped below the curing membrane and led to widespread scaling on the surface and limited spalling along joints.

Curing is crucial to concrete quality and at least twice it has been a primary factor in spalling and surface problems on USAF airfields. However, poor curing practices will always exacerbate any other problems that would contribute to spalling.

Volume change. Concrete undergoes constant volume change as it hydrates, shrinks, and changes in temperature and moisture content. Generally, conventional paving concrete mixtures occupy their greatest volume at the time of placement. Consequently, properly spaced transverse contraction joints cut within a few hours of concrete placement are normally sufficient to control cracking and accommodate normal volume change on airfield pavements. Older practices of placing wider expansion joints periodically in the pavement to accommodate volume change led to major maintenance headaches and were unneeded. Placement of periodic expansion joints along the length of a pavement was abandoned in the United States and many other areas over 50 years ago, though one may still observe such use of expansion joints in older pavements and sometimes overseas. Expansion joints today are used to isolate structures such as trench drains, refueling systems, building foundations, etc., where adjacent concrete pavement movement would cause problems.

If the expansion joint is not maintained properly or if it is not constructed properly, restrained movement can lead to spalling, as seen in Figure 5. Similarly, if contraction joints are not kept sealed and if incompressible debris accumulates in the joint, the debris-filled joint cannot accommodate the concrete expansion. The restraint against the concrete expansion can lead to spalling, as shown in Figure 5. Rollings and Rollings (1991) describe the development of widespread spalling on an Air National Guard taxiway that developed due to lack of joint sealant maintenance.



Figure 5. Examples of spalling from volume change restraint: (left) expansion joint material does not extend full depth of pavement, so concrete pavement movement causes spalling at the trench drain, Hurlbert AFB, Florida; (right) debris-clogged joints cannot accommodate temperature-induced movement in concrete pavement, leading to spalling, Middle Eastern airfield.

Freezing and thawing. In its *Guide to Durable Concrete*, Committee 201 of the American Concrete Institute (ACI 2001) notes that *exposing damp concrete to freezing-and-thawing cycles is a severe test for concrete to survive without impairment*. To achieve durability against freezing and thawing, concrete with severe exposure (such as pavement slabs) should

- *Limit exposure to moisture.* Freezing-and-thawing cycles will only damage critically saturated concrete; dry concrete is unaffected. Pavements are generally forced to survive under adverse moisture conditions for at least parts of their lives. The use of deicing chemicals on pavements accelerates the freezing-and-thawing damage process (ACI 2001).
- *Use a low water-to-cement ratio in the concrete mixture* (ACI 2001). For severe exposure such as pavements, the maximum recommended w/c ratio is 0.45 which is the same as required in USAF specifications.
- *Use sound aggregates.* Aggregates that are unsound when exposed to freezing and thawing can result in popouts (Figure 6) or more destructive durability-cracking (or D-cracking) (Figure 7). Both of these are potential FOD hazards and objectionable to the USAF, and spalling from severe D-cracking can generate a significant FOD hazard. USAF limits on deleterious materials that tend to lead to popouts are much tighter than most civil requirements, and this is essentially an attempt to limit FOD potential (Rollings et al. 2007).



Figure 6. Examples of popouts: (left) Laughlin AFB, TX; (right) Middle Eastern airbase



Figure 7. Examples of D-cracking: (left) severe spalling from D-cracking at Grissom AFB, IN; (right) moderate D-cracking with popouts, St. Joe ANG facility, MO.

- *Use entrained air.* This purposely entrained air consists of nearly spherical bubbles, 1 mm or less in diameter, which are dispersed throughout the paste. This finely dispersed network provides the critical reservoir of air-filled bubbles to accommodate the expansion of capillary water when it freezes. Entrapped air exists as bigger irregular bubbles and is too dispersed to be effective in protecting the paste from freezing. Consolidation with vibrators is used to remove as much entrapped air as possible from the concrete during placement. Entrained air normally remains stable and intact under normal construction placement and consolidation.

The air void system is generally considered resistant to freezing and thawing when the spacing factor, L -bar, as measured by American Society for Testing and Materials (ASTM) Method C 457 is 0.20 mm or less (e.g., Portland Cement Association 1988; ACI 2001, 2004; Neville 2002;

Kohn et al. 2003). This spacing factor (L-bar) is widely recognized as an improved tool to provide a better measurement of the air void system in the concrete, but it too has its limitations (Hover 2006). While the value of 0.20 is an often-quoted value for durability, L-bar values ranging from 0.152 to 0.25 have also been proposed (Jeknavorian 2006).

The USAF specifications for airfield concrete require 6% total air with a tolerance of $\pm 1.5\%$. This is adequate for severe exposure conditions for concrete mixtures with 1.5-in. maximum nominal size aggregate (ACI 2001). This is the most common maximum aggregate size used in USAF airfield concrete, but existing guidance would provide too little air for mixtures with smaller aggregate sizes.

Four runway and apron projects at Grissom AFB, five runway, taxiway, and apron projects at McConnell AFB, and one runway project at Randolph AFB all suffered extensive sliver spalling such as shown in Figure 8 (Rollings and Wong 1992; Rollings 1998; Rollings et al. 2008). Investigations found these pavements were low on air, had high spacing factors, and were vulnerable to freezing and thawing damage, even in mild climates. Figure 9 shows the relation between the spacing factor and occurrence of sliver spalling on these three bases and on one that had no sliver spalling for which data were available. This figure lends credence to application of the 0.20-mm criterion for identifying potential sliver spalling vulnerability in USAF airfields.

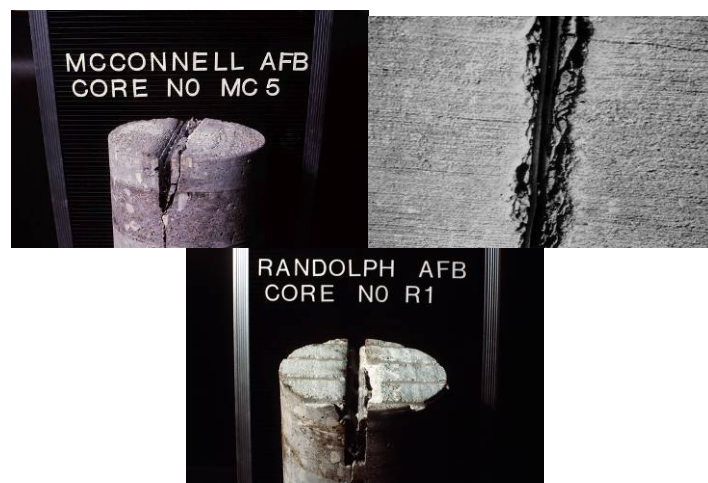


Figure 8. Examples of sliver spalls: (top left) McConnell AFB, KS; (top right) Grissom AFB, IN; (right) Randolph AFB, TX.

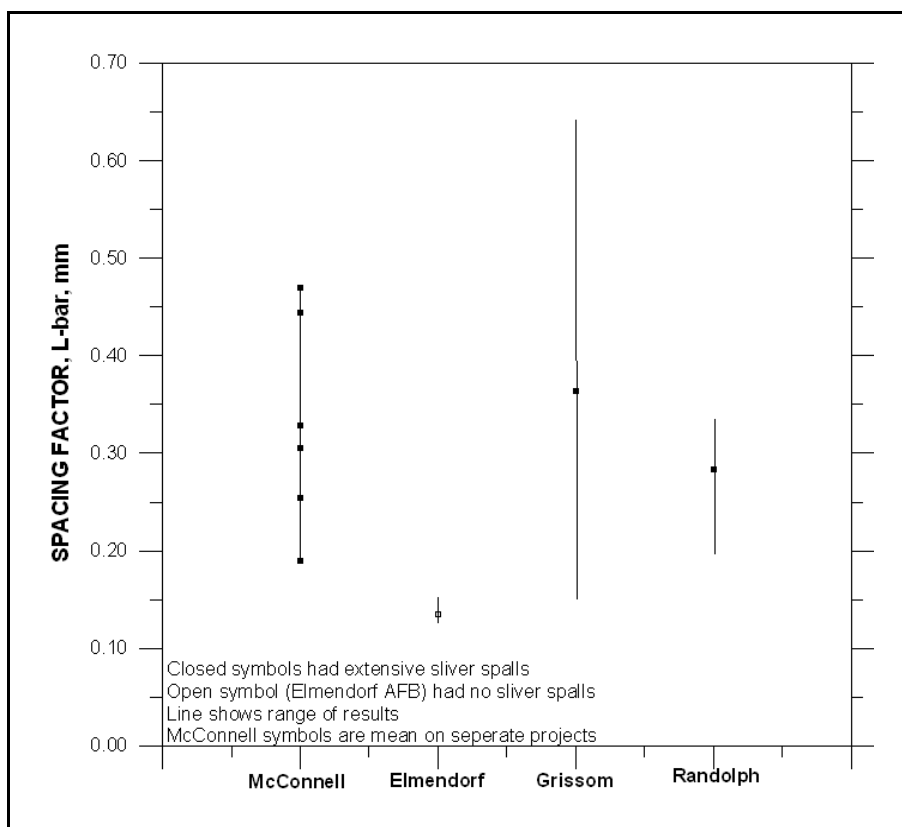


Figure 9. Relation between sliver spalling and spacing factor.

Alkali-silica reaction. The destructive expansive reaction between portland cement alkalis and certain aggregates (notably those containing certain forms of silica and to a lesser extent some dolomitic aggregates) is a well-recognized and much-studied aspect of concrete technology (e.g., ACI 1998a). The military had well-established procedures for checking for this potential problem and mandated use of low-alkali cements if reactive aggregates were to be used. However, by the late 1990s, nineteen USAF airbases had developed alkali-silica reaction problems with resulting problems with pavement expansion, spalling, popouts, and cracking. These problems with alkali-silica reaction reappeared for a variety of reasons, and the USAF now mandates specific aggregate testing and countermeasures as appropriate to avoid this problem in the future (AFCEA 2006).

The volume change inherent in alkali-aggregate gel formation and absorption of water puts the concrete pavement in compression (closing joints, heaving shoulders) and can damage adjacent structures, utilities, and pavement. Where these compressive forces are sufficiently large, pavement joints can spall. The pattern of cracking that normally also

accompanies this volume change can encourage this spalling to develop. An example of spalling from alkali-silica reaction expansion is shown in Figure 10.



Figure 10. Example of spalling from alkali-silica reaction compressing expansion joint causing spalling at trench drain, Holloman AFB, NM.

Corrosion. In the past, special metal tube inserts were sometimes used to form contraction joints and were left in place. These had a tendency to corrode and lead to spalling (Mullen and Fleggas 1972). These have not been used in the USAF in decades, although there may still be some old pavements with these inserts.

USAF early-age spalling problems

Generally, spalling develops later in the life of the pavement and may arise for a number of reasons—including durability problems (e.g., D-cracking or alkali-silica reaction), accumulation of incompressible material in the joints, corrosion of embedded steel, or deterioration around cracks and

joints from loading (Mullen and Fleggas 1972; Belangie 1990; Hodges 1990; Rollings and Rollings 1991; Markey et al. 2005; AFCESA 2006, 2007). However, the USAF has been plagued periodically by early-age spalling on airfield pavements wherein spalls form within the first year after construction, sometimes even appearing before construction has finished. Because these spalls pose a potential FOD hazard on essentially brand-new pavements, they cause major complaints from the flying units. The most common and troublesome of these early-age spalls include sliver spalls (Figure 8) and failure of improper edge-slump repairs (Figure 4).

Table 1 provides a summary of early-age spalling and surface defect problems at USAF bases from 1981–2005. Of the 15 bases studied, snow-clearing operations were the primary cause of damage in 13% of the cases, grinding operations in 7%, curing in 13%, inadequate air entrainment in 13%, and construction issues in 54% of the cases. As in most field failures, there are often multiple contributing causes for any one specific case although, clearly, construction defects are the biggest single contributor to early-age spalling in USAF experience. In many cases, poor construction

Table 1. Causes of early-age spalling on USAF projects built 1981–2005 (Rollings et al. 2008).

Airbase	Primary Contributor	Secondary Contributors
Cannon AFB, NM	Snow clearing	Grade changes
Ellsworth AFB, SD	Construction practices	Inadequate air entrainment
Elmendorf AFB, AK	Curing	Early loading
Grissom AFB, IN	Inadequate air entrainment	
Hector Field, ND	Construction practices	Inadequate air entrainment
Kelly AFB, TX	Construction practice	Concrete mixture
Laughlin AFB, TX	Grinding operations	
Malmstrom AFB, MT	Construction practice	Aggregates
McConnell AFB, KS	Inadequate air entrainment	Concrete mixture
Myrtle Beach AFB, SC	Snow clearing	
Randolph AFB, TX	Construction practices	Inadequate air entrainment
Tonopah Test Range, NV	Construction practices	
Travis AFB, CA	Construction practices	Unsound aggregates, Concrete mix
Whiteman AFB, MO	Construction practices	Concrete mixture, Inadequate air
Wright-Patterson AFB, OH	Curing	

Sources: Rollings and Wong (1992); Rollings (1998); Rollings et al. (1998); Rollings et al. (2002), Rollings (2005); Rollings and Rollings (2005); Rollings (2006); and unpublished data.

practices (adding water to the surface during finishing, overfinishing, edge-slump repairs with a slurry, etc.) result in localized areas of weak concrete or surface paste with inadequate air entrainment.

Offutt AFB runway and taxiway problems

Between June and mid-October 2006, portions of Runway 12-30 and Taxiways (TW) C and P were reconstructed at Offutt AFB. The new replacement portland cement concrete pavement was 21 in. thick. Slabs on the runway matched existing slab dimensions, were 25 ft by 25 ft in plan, and were reinforced with 0.05% steel. The reinforcing was required because current USAF criteria do not allow unreinforced concrete airfield pavement slabs with 25-ft dimensions. Such slabs were allowed and common under old USAF criteria from about 1945 through 1984. The slabs on the taxiways are unreinforced and are 18.75 by 18.75 ft in plan.

The U.S. Army Engineer District (USAED), Omaha, was responsible for project design, construction contract management, and government quality assurance. Hawkins Construction Company of Omaha, NE, was awarded the construction contract. Runway 12-30 keel replacement was done in the summer and early fall 2006, and the taxiways were placed in September–October 2006. Construction apparently went relatively smoothly, and no significant defects or problems were identified during construction. Figure 11 shows the project site.

In March 2007, Offutt AFB personnel began to observe spalled joints and some cracking on the new runway and taxiway pavements. This spalling and incidence of cracking has reportedly continued to develop and worsen over time. A significant amount of debris is produced by this spalling and is a potential foreign object damage (FOD) hazard (Figure 11, inset).

Preliminary assessments of this damage by various parties failed to reach consensus on the causes of the problem (Poole et al. 2007). The study reported in the remainder of this report was conducted specifically to examine the damage at Offutt AFB in depth, to compare this performance to other airfield pavements and available data in the technical literature, and to develop recommendations for avoiding this problem in future projects.

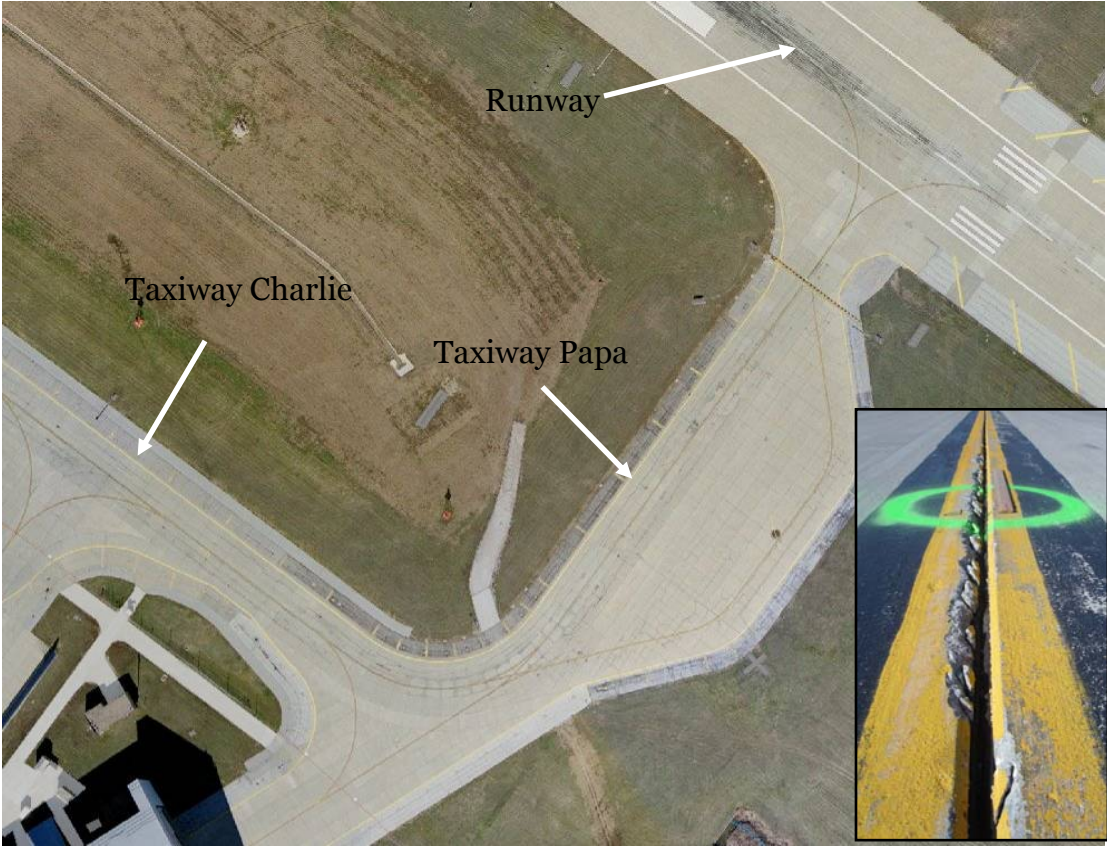


Figure 11. Study site at Offutt AFB with an example of damage inset.

2 Samples, Test Methods, and Auxiliary Data

General

The concrete at Offutt AFB that was placed in the summer and fall of 2006 has been the focus of the data collection and analysis for this investigation. This is the same concrete that contained the damaged joints which prompted the study. Three cores were also received from Eppley Field, which is the commercial airfield in Omaha, NE. This concrete was placed at about the same time, by the same contractor, using similar mixture designs. This concrete is reported to not have suffered the joint problems seen at Offutt.

Samples 1–14, taken in spring 2007 and the object of the analysis reported in Poole et al. (2007), were relatively shallow, representing approximately the top 200 mm of the pavement. Following review of that report, it was determined that full-depth cores were needed to allow fabrication of specimens for testing for durability to cycles of freezing and thawing, and also to provide some additional data on air void properties of the concrete.

A site visit was made to Offutt AFB on 13 Sep 2007 for the purpose of taking cores for the investigation. Fifteen full-depth cores were taken. The rationale for taking the various cores varied. Table 2 summarizes the locations and other notes on the samples.

Samples were also obtained from Mountain Home AFB, ID, and from Eppley Field in Omaha, NE. Limited testing was done on these for comparative purposes.

The Mountain Home samples were from old concrete being demolished and were secured because this concrete seems to have served very well. The plan was to analyze entrained air parameters for comparative purposes.

Table 2. Notes on coring performed 13 Sep 07 at Offutt AFB.

Core No.	Location ¹	Notes
15	RW	70 mm left of centerline (CL) ~6 ft back from Core 12. 7-mm wire 13 cm from top. Full depth 55 cm, 14.5-cm diameter. Replicate left slab adjacent to Cores 11 and 12.
16	RW	59 cm right of CL between loc of Cores 11 and 12. More damaged side of the joint. No. 2 reinforced steel ~5.5 in. from top. Replicate right slab adjacent to Cores 11 and 12.
17	RW	Same location as Core 13, on joint 110 cm down. Const joint. No machine-formed bevels. Full depth (35 cm on shoulder side, 37 cm on RW side). Core 13 was lost during shipment for earlier analysis.
18	RW	Same location as orig. Core 13, but 52 cm into RW side of the joint. Core is 36 cm thick. Represents the more damaged side of the joint.
19	TWP	Core through corner damage (series of ~random cracks, incipient large spall).
20	TWP	Core through corner damage.
21	TWP	Popout in center of slab. 56 cm thick.
22	TWP	53 in. away from orig. Core 10. Replicate Core 10.
23	TWC	Near orig. Core 3. Original Cores 2 and 3 are w/in ~5 ft of each other.
24	TWC	Near original Core 2.
25	TWC	Old concrete (1983). Cracking pattern suggests alkali-aggregate reaction. Cracks to 40 mm deep.
26	TWC	Near original Core 4. 55 cm thick.
27	TWC	Near original Core 1 (TW side).
28	TWC	Near original Core 1 (asphalt side).
29	TWC	Through the apparent tears resulting from saw.

Note: Samples 1-14 referenced in the notes are from the sampling of spring 2007 and are the basis for the analysis reported in Poole et al. (2007).

¹RW = runway; TWP and TWC = Taxiways P and C.

The Eppley Field samples were secured because this concrete was reported to have been placed at about the same time as the Offutt concrete, using largely the same materials and mixture proportions. The coarse aggregate source was different. The same deicing salts were reported to be used as at Offutt, but different snow-removal techniques were reported to have been used at the two sites.

Field monitoring of damage

A comparison of joint damage during winter 2007–2008 was executed using photographic techniques. The site was visited on 19 Dec 2007, and two sections of concrete were selected for monitoring of joint damage (diagrams of monitored sections shown in Figures 12 and 13). Pre-winter photographs were taken.

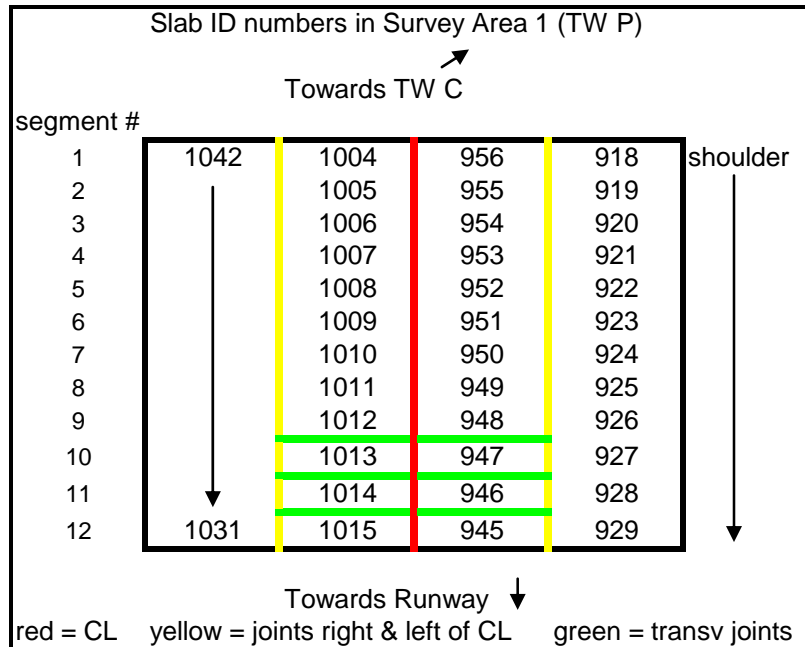


Figure 12. Joint and slab identifications, monitored section of Taxiway P.

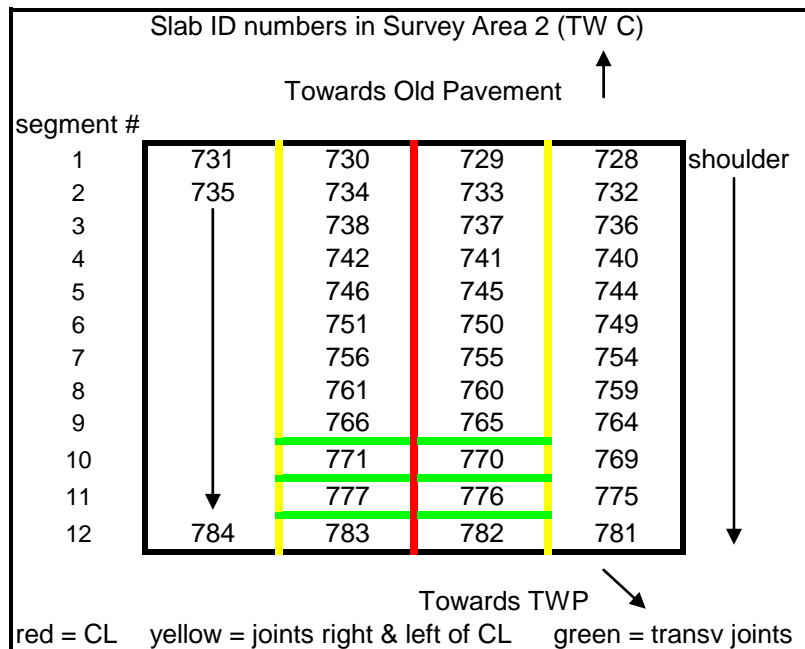


Figure 13. Joint and slab identifications, monitored section of Taxiway C.

Photographs were taken from a tripod at a height of about 1.5 m. Each 20-ft section of joint was photographed from each end. The camera was a Canon A630 8 mega-pixel model with resolution set at 3264×2448 pixels. The exposure time and F-stop setting were determined by the automatic feature of the camera. The resolution and zoom were such that approximately the first half of the joint was visible on the resulting photographs. This was found to be sufficient detail to resolve damage features. Since each joint was photographed from each end, the entire joint section was visible. The site was visited again on 10 Jun 2008, and the operation was repeated.

Durability to cycles of freezing and thawing – ASTM C 666

ASTM C 666 was the primary test method used to evaluate durability of concrete to cycles of freezing and thawing. This method recognizes the use of field cores in this test, although they must be cut to the size requirement of the method (3.5×4.5 in. in cross section). To stabilize during sawing, the cores were mounted in a wooden frame and mechanically stabilized for sawing by embedding the bottom one third in plaster of Paris. The cylindrical specimens were then sawed into rectangular cross-section specimens using a variable-feed-rate 60-in. concrete saw mounted on an overhead feed beam. The saw was manufactured by Sawing Systems, Knoxville, TN. Care was taken to feed the saw into the specimen very slowly so as to achieve clean, accurate cuts with minimal collateral damage. These specimens contained limestone coarse aggregate, which is relatively easy to saw.

Several variations in procedure are allowed by C 666. All have in common that specimens are cycled between -18 and $+4$ °C. The ERDC rendering of the method uses rectangular steel cans to hold the specimens. The interior dimensions of these cans are slightly larger than the specimens (within the 1- to 3-mm gap required of the method). Specimens were continuously surrounded by water (Method A). Freezing and thawing are accomplished by alternately circulating cold and warm antifreeze solutions around the steel cans. Each freezing and thawing cycle is executed in 2 h. The specimen chamber was cycled between 4 and -18 °C in 2 h, so that 12 complete freezing-and-thawing cycles are executed in 24 h. This is the fastest cycling rate allowed by C 666.

Specimens were removed from the freezing and thawing apparatus approximately every 3 days, and resonant frequency and mass

determinations were made. The specimens were then inverted and returned to the freezing chamber. This inversion is important because the conditions in the bottom of the can tend to be somewhat harsher than in the middle and upper parts. This practice of alternating ends results in some averaging of the effect between the two ends of the specimen.

In this method, cycling continues until the specimens either reach 300 cycles or the relative modulus of elasticity (Rel E%) drops below 50%. Some specimens were carried beyond the 50% cutoff in order to highlight damage features.

Air void analysis – ASTM C 457

Air void parameters were determined according to ASTM C 457, using the point-count procedure. Pertinent air void parameters include total air (%), air void spacing factor (mm), and air void specific surface (mm^{-1}). The method also gives estimates of paste volume (%) and aggregate volume (%).

Effects of potassium acetate (KAc) deicing salt

ASTM D 5312 was used to investigate the effects of potassium acetate (KAc) deicing salts on durability to freezing and thawing. This test method is designed to test rock for durability to cycles of freezing and thawing, and was adapted to this analysis.

In this method, a piece of rock (or concrete) is placed on a bed of absorbent material in a pan. Water is placed so that it keeps only the absorbent material soaked. This results in only one surface of the concrete being critically saturated in the test.

Temperatures were cycled between -18 and 40 °C over a 24-h period. Heat transfer medium is air, so that the rate of freezing and thawing is somewhat slower than in C 666, in which heat transfer is by a liquid.

In this analysis, the effects of potassium acetate are evaluated by first soaking the chosen surface of the concrete specimen in a 50% KAc solution for 7 days at 38 °C. Then, the specimen is removed from the salt solution and placed in the test apparatus containing water. The only KAc remaining in the test at this point is whatever is soaked into the concrete, which is free to diffuse into the water when the test starts.

For purposes of the analysis, tests were run no longer than 20 freezing and thawing cycles. Specimens were removed every few days to observe and photograph any developing damage.

Aggregate mineralogy

The mineralogy of coarse aggregate particles showing evidence of damage due to cycles of freezing and thawing was determined using X-ray diffraction (XRD).

Mineralogical analysis of samples is done using XRD analysis on random powders or oriented samples using standard techniques for phase identification. The equipment used in these analyses is a Philips PW1800 Automated Powder Diffractometer system. The run conditions include the use of CuK α radiation and step scanning from 2 to 65° 2 θ with 0.05° 2 θ steps, and collecting for 3 to 4 sec per step. The collection of the diffraction patterns is accomplished using PC-based windows versions of Datascan (Materials Data, Inc.) and analysis using Jade.

Low-power microscopy

Low-power microscopy was used both for C 457 analysis and for examining the paste and aggregate fractions of concrete for evidence of damage from laboratory exposure to cycles of freezing and thawing, and for effect of KAc deicing salts. Microscopes were Nikon Model SMZ-U. Magnifications ranged from 5 to 35 \times for general observation, and 50 \times was used for the C 457 analysis.

Sample preparation varied, depending on the intention of the procedure. Sometimes samples were examined with no sample preparation at all—simply observation on the surface effects of the treatment. Detailed examination of data in selected areas was on thin sections. Polished surfaces, as in C 457, were used for certain other examinations.

Thin section preparation was outsourced to National Petrographic Services, Houston, TX. Samples were ground in water and impregnated with a colored epoxy to provide contrast for observation of cracks and void distribution patterns.

Construction and other pertinent data

Data on placement dates, total air content measurements (as delivered to the pavers), and other details of the paving operation were provided by personnel of the USAED, Omaha, who provided the project quality assurance (QA). These individuals also provided information on activities and observations subsequent to the construction.

The USAF provided maps showing extent of damage following winter 2006–2007, along with observations on the development of the damage, pavement management, and maintenance activities.

Individuals working in areas near Offutt AFB were interviewed for information that might be pertinent to the problem. These included a retired employee of the Kansas Department of Transportation (DoT), a representative of the American Concrete Paving Association (formerly of the Iowa DoT and residing in western Iowa), a representative from the Nebraska DoT, and an employee of the Iowa State Center for Concrete Pavement.

Maturity calculations

Maturity calculations were executed using the equivalent age maturity function found in ASTM C 1074. This calculation requires determination of a constant, Q . A default Q value of 5,000 K is recommended by the test method when it is not practical to determine the constant for a particular application. That was the case here. Also, temperature versus time data for the concrete is required, along with a reference strength-gain curve for the concrete under known temperature conditions.

The assumption was made that the concrete temperature reasonably tracked the average air temperature. This probably is not true during the first day or two because of the buildup of heat in the relatively thick pavement due to the heat of hydration of the cement. Therefore, strength gain is probably a little faster at early ages than calculated in this analysis. Average daily air temperature data from Lincoln, NE (~50 miles away) was used to approximate the in-place concrete temperature. The strength versus time data in the Thiele Geotech preconstruction concrete report was used as the data for reference strength-gain curve. It was assumed that the temperature for these data was close to 23 °C, which is standard laboratory curing temperature.

3 Results and Commentary on Results

ASTM C 666 – Durability to Cycles of Freezing and Thawing

The purpose of conducting the ASTM C 666 was to give a controlled measure of the concrete's resistance to cycles of freezing and thawing when critically saturated with water. This was originally planned to be the response variable for evaluating the effectiveness of the air void system in the concrete. As will be seen at the end of the analysis, the problem was determined to be much more complicated than a simple air void deficiency.

Relative dynamic modulus of elasticity versus number of cycles of freezing and thawing

Samples from Offutt AFB and Eppley Field were tested according to C 666. The change in relative dynamic modulus of elasticity (Rel E) with increasing number of freezing and thawing cycles is shown in Figures 14 and 15.

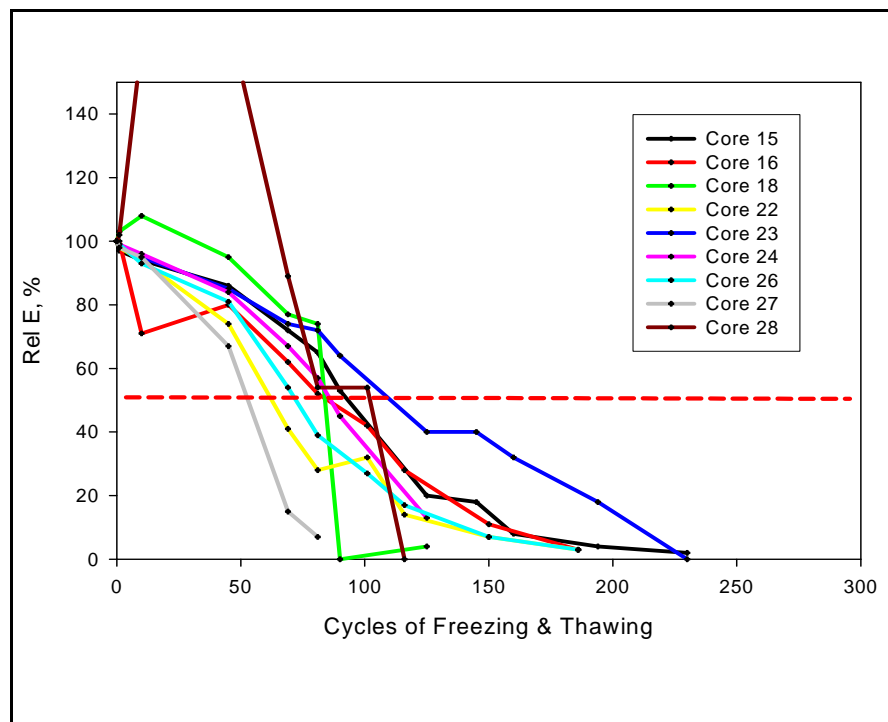


Figure 14. Relative modulus of elasticity versus number freezing and thawing cycles for nine specimens from Offutt AFB.

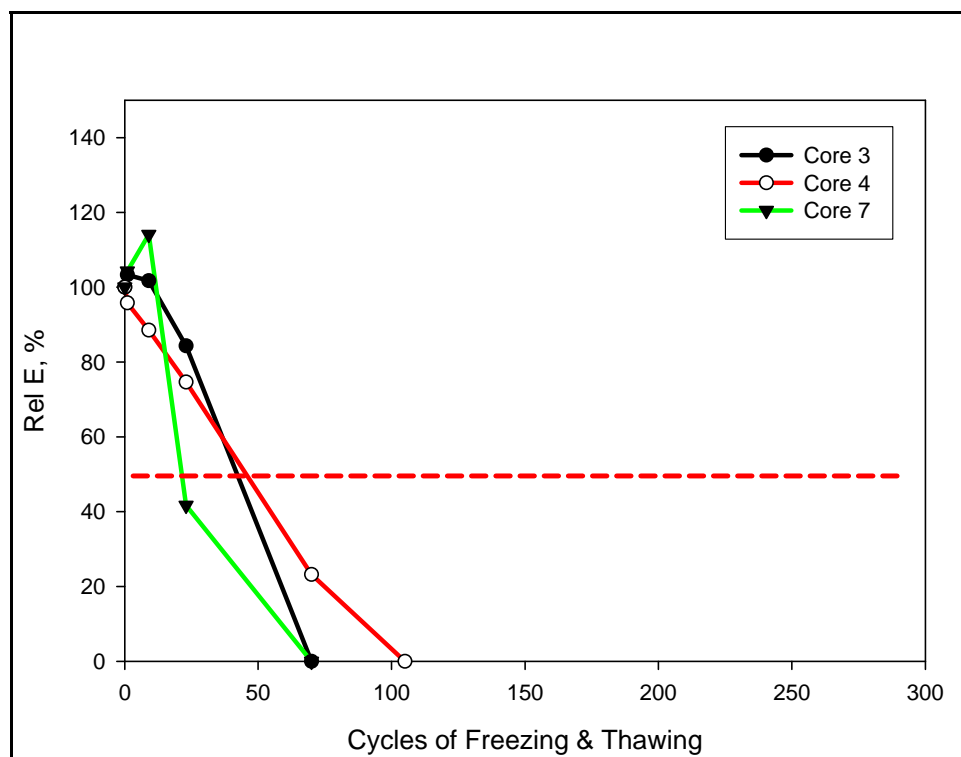


Figure 15. Relative modulus of elasticity versus number freezing and thawing cycles for three specimens from Eppley Field.

In general, the specimens from both of these projects did not perform particularly well in the C 666 environment. In both cases, the Rel E fell below 50% within 50 to 100 cycles of freezing and thawing. This would not be considered durable concrete by most standards. However, as will be presented and discussed below, interpretation of these results is not simple.

Post-test analysis of the C 666 specimens revealed at least three mechanisms of damage: expansion in large water-filled voids; nondurable coarse aggregate; and damage to the cement paste fraction. Some of this damage may be artifacts of the laboratory method, while other damage appears to be evidence representing plausible field damage.

Damage from water-filled voids

A notable early test damage feature in the C 666 specimens was the development of rather large popouts. Several of these are shown in Figures 16–18. At the base of these popouts, and presumably the cause of them, is usually the remnant of a 5- to 10-mm void that was located within about 10 mm of the surface of the specimen.



Figure 16. Specimen 26 after 81 cycles of freezing and thawing.



Figure 17. Eppley Core 3 after 70 cycles, showing a circumferential crack at least partially affected by subsurface voids. The arc-shaped damage feature in the lower center is a spall over a subsurface void that is fracturing off.

When these subsurface features are in apparently critical proximity, a crack is often observed linking them together and causing a major damage feature, which results in the failure of the specimen in the test method. An example of this damage is shown in Figures 16 and 17.

The early evidence of this form of damage was evident in the earliest visual inspections, at about 25 cycles. The rate of formation of new popouts seemed to diminish after about 75 cycles.

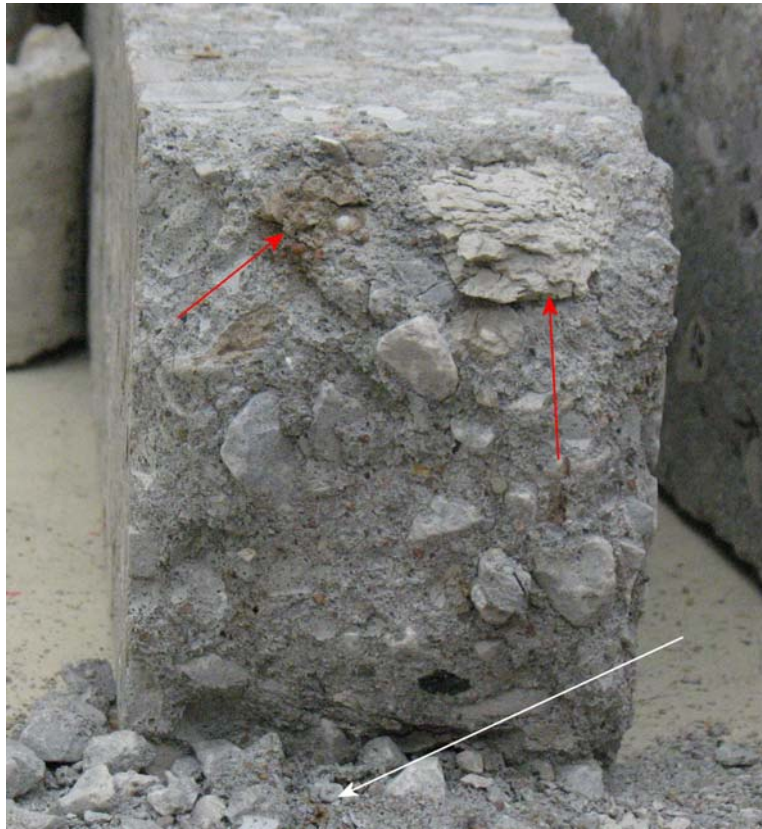


Figure 18. Damage to Offutt sample 26, as shown in 45 x 115 mm cross section, after 186 freezing and thawing cycles. Red arrows indicate two severely damaged aggregate particles. White arrow shows aggregate and crumbled paste that has fallen off of the specimen, typical of damage to the cement-paste fraction of the concrete.

The damage caused by filling and subsequent freezing of subsurface voids is probably more a feature of the test method than a major feature of the damage in the airfield concrete. Specimens are always immersed in water in C 666, so that after a certain period of time, these voids plausibly become full of water. There is no place for this water to drain, since the concrete underlying the void is also saturated. The resulting expansion during ice formation in the void results in a relatively violent damage event, similar to the condition that results when a water-filled glass bottle is frozen.

In field concrete that is saturated from one or, at most, two sides, water in the voids can drain away toward the nonsaturated concrete. Also, environmental saturation sufficient to fill large voids in in-place concrete is not a constant feature, but rather an intermittent, with intervals of drying.

It is likely that much of the early rapid declination of Rel E values is attributable to this source of damage. Therefore, the simple plots of Rel E versus number of cycles of freezing and thawing may not reveal a great deal of information about field durability.

Damage from unsound aggregate

Visually detectable damage to aggregate particles was noted in the first visual inspection of the Offutt samples after 81 cycles of freezing. Results from another freezing-and-thawing test (D 5312), reported below, revealed damage to one particular type of aggregate after less than 20 cycles. Aggregate became increasingly damaged with increasing number of cycles. Figure 18 shows an example of conspicuously damaged aggregate after 186 cycles. The Eppley samples also contained damaged aggregate, but there seemed to be less of it, and the damage tended to develop at somewhat higher numbers of freezing and thawing cycles.

Damage to the cement-paste fraction

When damage occurs in the cement-paste fraction of the concrete, it first appears as microcracks among the fine aggregate, then as severe crumbling of the concrete as the cracks work into the cement paste-aggregate interface, leaving aggregate particles to fall loose. This can also be seen in Figure 18.

It is a reasonable interpretation that this severe damage indicates zones of poor air entraining in the concrete, although unsound aggregate could accelerate the phenomenon by increasing permeation of water. While such damage more commonly appears at the ends of the specimen where the intensity of freezing events is higher, it is also seen in other locations (e.g., Figure 19).

Gross damage to cement paste was shown in Figures 18 and 19, along with aggregate damage. Another more subtle form of damage was a general etching of the cement paste, without apparent gross damage to the specimen. Figures 20 and 21 present a comparison of specimen 15 after 125 and 230 cycles of freezing and thawing, respectively. The depth of the damage to the paste is approximately 1–2 mm.



Figure 19. A specimen from Eppley Field after 230 cycles. The red arrow points to a zone of concrete that is deficient in coarse aggregate and probably also air. The blue arrow points to a deeply damaged zone that started as damage from a 10 mm entrapped air void and later enlarged, apparently due to paste damage. Note that the concrete, and specifically the paste, on the right-hand end of the specimen is in good condition.



Figure 20. Cement paste fraction of sample 15 after 125 cycles of freezing and thawing. Corners are relatively sharp and paste etching is minimal.

In general, the gross damage to the paste tended to appear rather late in the testing regimen. It was particularly noticeable after 200 cycles of freezing and thawing, although there were examples of severe small-scale paste damage after fewer than 100 cycles, as described below.



Figure 21. This photograph shows the same area of sample 15 shown in Figure 20, after 230 cycles. The paste is still largely intact, as indicated by the still relatively sharp edges, although the aggregate is clearly undercut due to loss of cement paste.

Small-scale damage

Examination of intact samples using low-power microscopy and in thin sections made from three C 666 specimens removed from testing after 81 to 125 cycles showed evidence of small-scale damage apparently stemming both from unsound aggregate and air-entraining deficiencies. Unsound aggregate appears as cracks within the aggregate that normally propagate out into the paste. Some of these cracks will then propagate around the cement paste-aggregate interface. Damage from air-entraining deficiencies normally appears as small cracks running among fine aggregate particles. These also open up cracks along the cement paste-aggregate interface. The cement paste-coarse aggregate interface is a particularly weak zone in concrete and ultimately loses integrity in many forms of concrete damage.

Figure 22 is a low-power view of a section of sample 27 after 81 cycles of freezing and thawing. The fine-scale cracking among the fine aggregate is clear here. This damage, along with the adjacent void, was part of a circumferential crack that probably caused the precipitous decline in Rel E in this specimen starting very early in the testing.



Figure 22. Paste damage in sample 27 after 81 cycles of freezing and thawing.

Figures 22–29 illustrate these patterns in thin sections made from C 666 specimens. Figures 30–33 show examples of field concrete taken in May 2007 that had been exposed to one winter’s exposure. The blue and yellow color differences in these photographs are due to different colors of epoxy used in the preparation.

Figure 23 shows a badly damaged section of Core 15 after 125 cycles of freezing and thawing. The damage is almost totally in the paste fraction. Figure 24 shows a thin section taken from a section of sample 27 at a sawed surface after 81 cycles. Light surface etching of the cement paste is visible at the top. Cracking is evident in the coarse aggregate particle, along with some very fine cracking in the cement paste, indicative of air problems. The paste fraction does not appear to have a lot of entrained air voids. Similar thin sections from sawed, but unexposed, concrete do not show this type of microcracking, discounting damage as an artifact of sawing. Figure 25 shows an example of damage in the paste fraction only. This thin section does not contain a lot of entrained air. This sample was also taken from the C 666 specimen cut from Core 27 after 81 cycles of freezing and thawing.

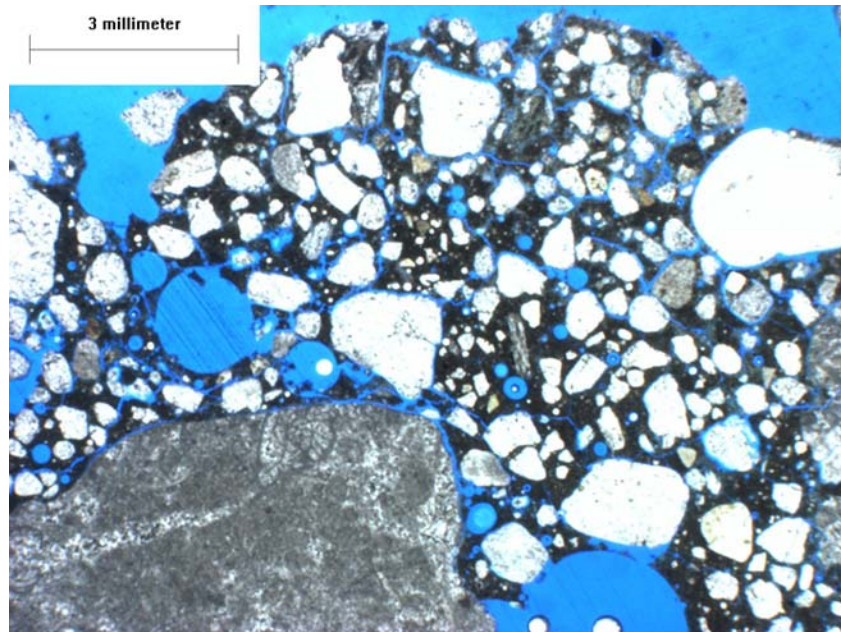


Figure 23. Badly damaged fragment of Core 15 after 125 cycles. Note the many very fine cracks running among the fine aggregate particles, particularly in the upper-right quadrant of the photograph.

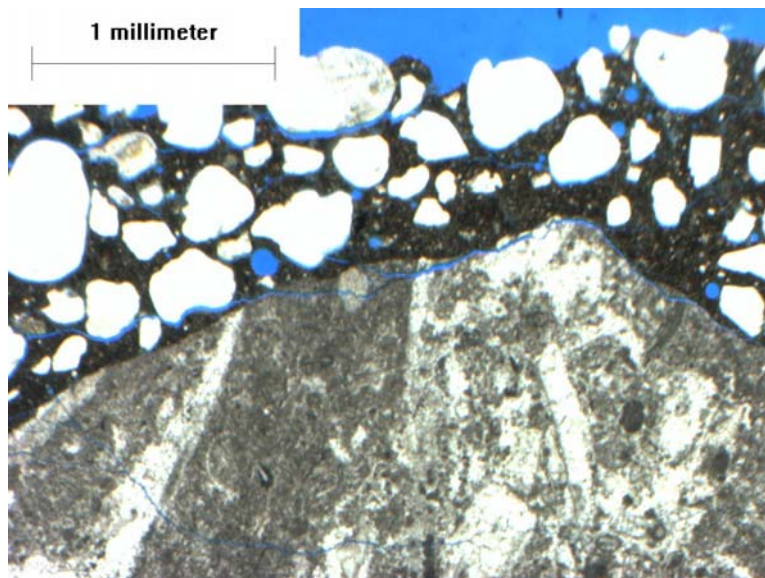


Figure 24. Taken from the edge of C 666 specimen from Core 27 after 81 cycles of freezing and thawing. Cracks in the large aggregate particle at the bottom of the photo are evident. The fine cracks in the paste fraction at the top of the photo indicate paste damage due probably to insufficient air voids.

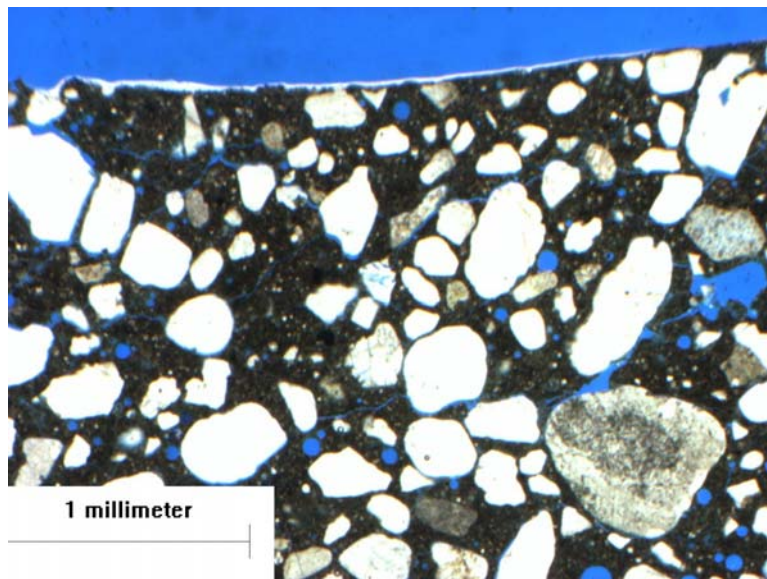


Figure 25. Damage to paste fraction.

Figure 26 is taken from the C 666 specimen cut from Core 18 after 125 cycles of freezing and thawing. Damage is dominated by aggregate cracking, although there is some cracking that may have an air-entraining causation.

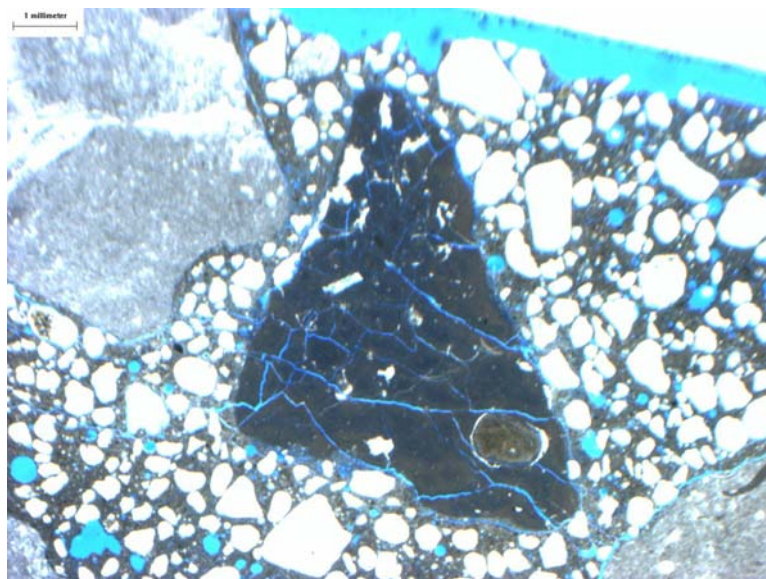


Figure 26. Specimen from Core 18. Aggregate damage is conspicuous and continues into the cement paste.

Figure 27 is also taken from the Core 18 specimen after 125 cycles of freezing and thawing in an area of gross damage in the form of crumbling concrete. Both aggregate damage and paste damage are conspicuous. The cement paste-aggregate interfaces are seriously opened. This sample appears to have damage from both unsound aggregate and poor air entraining.

Figure 28 features a piece of coarse aggregate in sample 18 after 125 cycles that was showing significant visual damage. The damage removed the material from the exposed surface, but the cracking did not appear to penetrate into the concrete below the aggregate particle.

Figure 29 is a thin section from Core 24 after 125 cycles showing significant cracking in the coarse aggregate particle propagating into the cement paste.

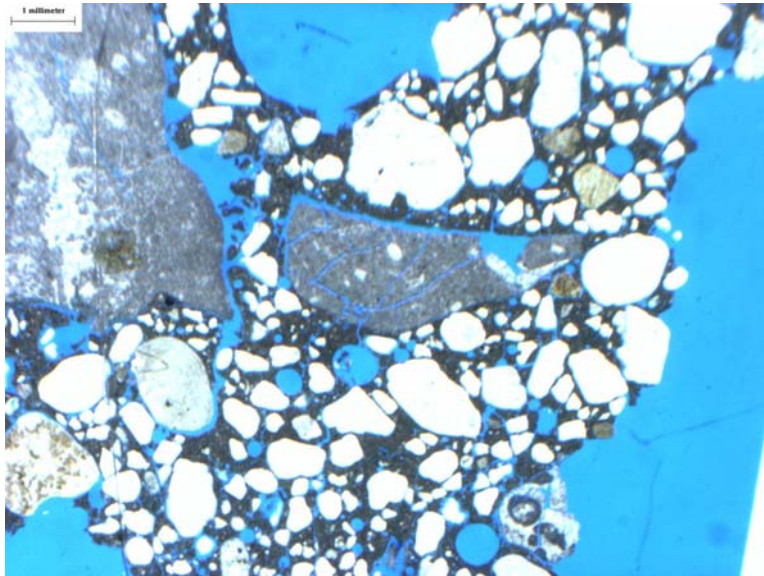


Figure 27. Thin section of a serious damaged part of the C 666 specimen from Core 18.

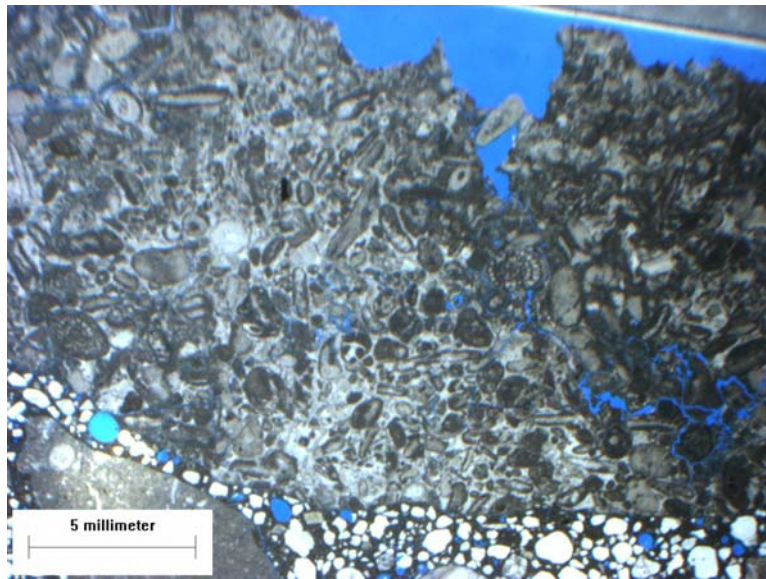


Figure 28. Damage to a piece of coarse aggregate in C 666 specimen from Core 18.

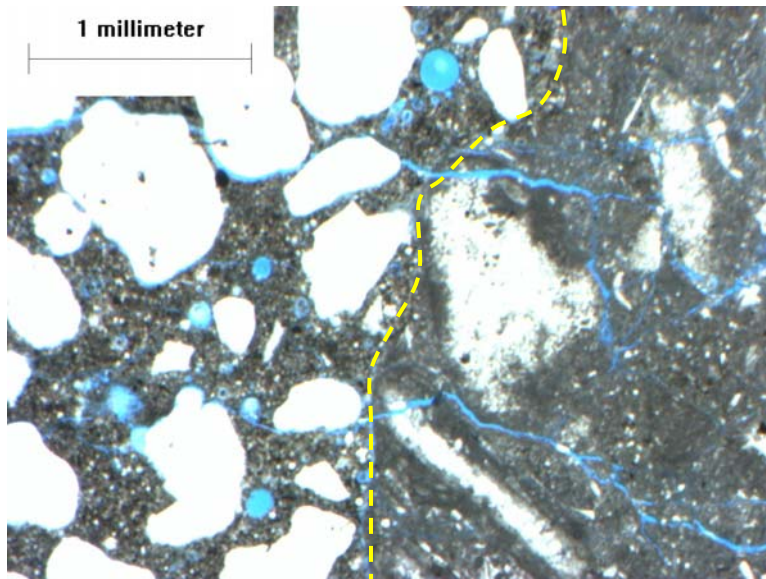


Figure 29. A relatively clear example of aggregate damage propagating into the cement paste fraction of a C 666 specimen cut from Core 24. The aggregate particle is the material to the right side of the yellow line.

Thin sections made from concrete samples taken from field-damaged concrete showed damage that resembles the damage seen in the C 666 samples. Examples of these are shown in Figures 30–33. Figure 30 is a cross section of a sliver spall. Two fine cracks appear to originate in the piece of aggregate at the joint-edge bevel and to propagate downward parallel to the sawed joint-seal recess.

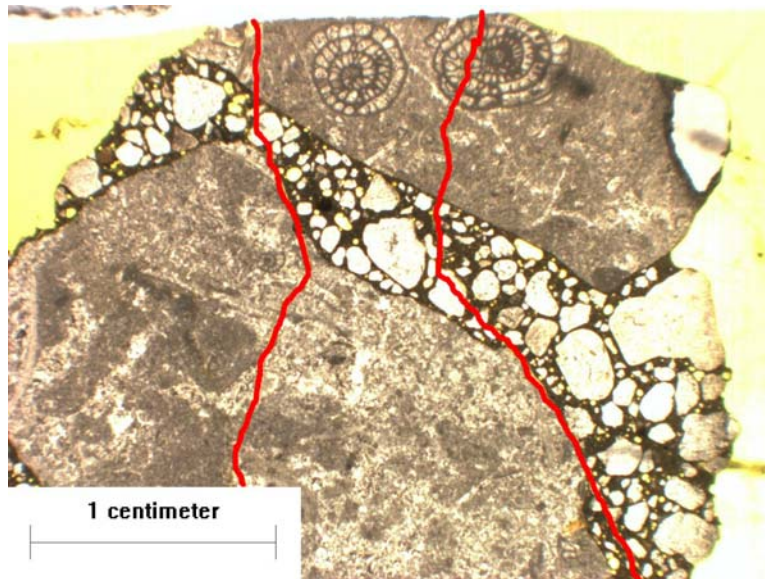


Figure 30. Cracking in a sliver spall taken from a damaged section of centerline joint. The cracks were very fine and have been colored red to highlight their path. The beveled surface is at the top of the picture and sawed surface is to the left. The fractured surface that caused the separation is to the right.

Figure 31 shows an example of what appears to be cracking originating from a piece of aggregate. This sample was taken from concrete next to a sawed joint that had some damage.

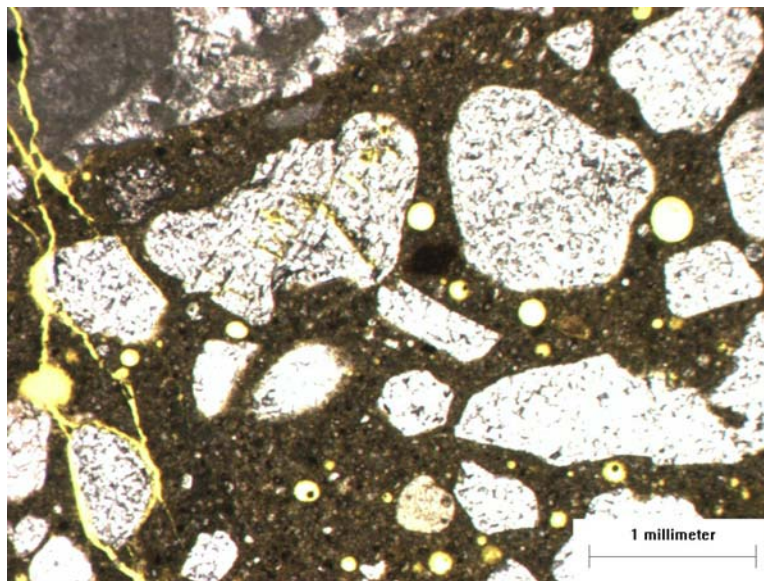


Figure 31. Aggregate-induced cracking in a sample of field exposed concrete taken from near a damaged sawed joint. The coarse-aggregate particle is in the top-left corner.

Figures 32 and 33 show micro-scale cracking in a section of field-exposed concrete, which appears to have very little air entraining—particularly in Figure 32.

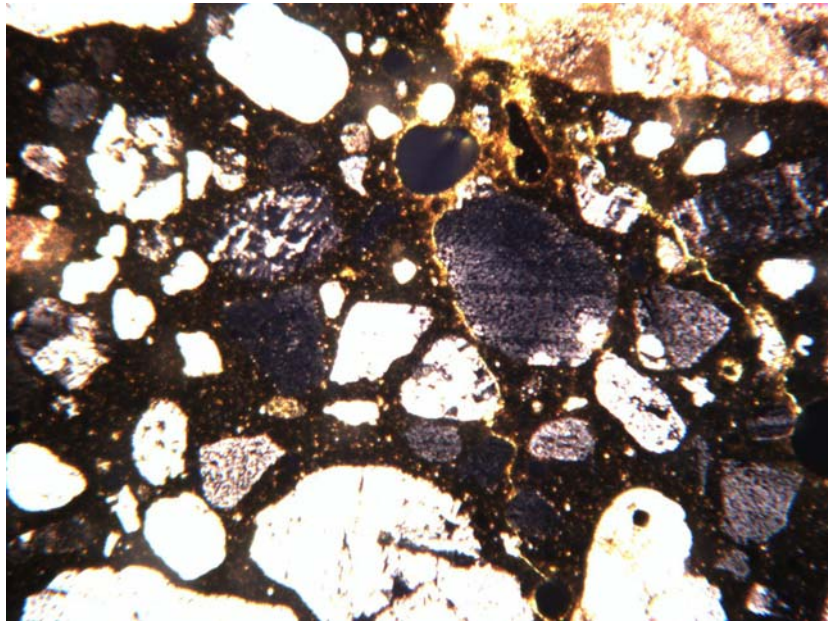


Figure 32. Apparent damage due to a small section of poor air entraining.

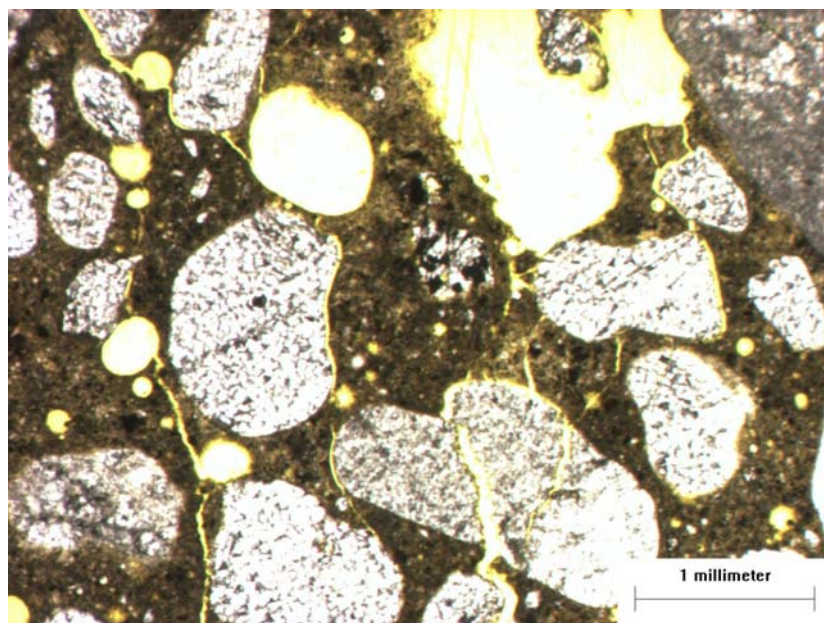


Figure 33. Cracking that resembles the type seen with poor air entraining.

Effects of the deicing salt potassium acetate

The original project plan was to conduct freeze-thaw testing with deicing salts as part of the exposure condition. The specific methodology was not detailed. It was determined that C 666 would be difficult to adapt to this purpose. Another test, ASTM D 5312, which is used by the ERDC to evaluate the durability of rock to cycles of freezing and thawing, appeared to lend itself better to this application.

Sections of five of the cores taken at Offutt were sawed into two companion specimens. Large pieces of aggregate that could be matched across the sawed joint were noted for future reference, in case any showed evidence of deterioration in the test. One set of five specimens was placed in a shallow pan of 50% (w/v) potassium acetate solution for 1 week at 38 °C so that one side was exposed to the solution. The companion control specimens were soaked in water. Specimens were removed from this conditioning and tested according to ASTM D 5312. Specimens were removed from the test every few days (one cycle of freezing and thawing per day) and examined for evidence of damage.

Damage was evident in some of the specimens soaked in the potassium acetate solution after eight cycles of freezing and thawing, which was the time of the first inspection. Damage occurred both in the paste and in some aggregates. The damage continued to accumulate with increasing number of cycles. The tests were run for 19 cycles of freezing and thawing. At this point, the paste fraction of all specimens was deteriorated to a depth of about 1 mm. The coarse aggregates that were sensitive to the effects of the presence of the salts also continued to show increasing damage throughout the test period.

Control specimens showed almost no damage during this test. One exception was a very light-colored coarse aggregate that tended to show evidence of damage from freezing and thawing, even under the control condition.

Figures 34–38 show examples of typical damage to aggregate and paste fractions of a specimen exposed to potassium acetate after eight cycles of freezing and thawing.



Figure 34. A 10-mm aggregate particle showing first signs of damage after soaking in potassium acetate and then exposure to eight cycles of freezing and thawing in water.



Figure 35. Mortar fraction of Core 16 after soaking in potassium acetate and then exposed to eight cycles of freezing and thawing, showing exposure of aggregate particles due to erosion of paste.



Figure 36. An example of paste etching and aggregate damage (along the left edge) after 19 cycles of freezing and thawing in water after exposure to potassium acetate.

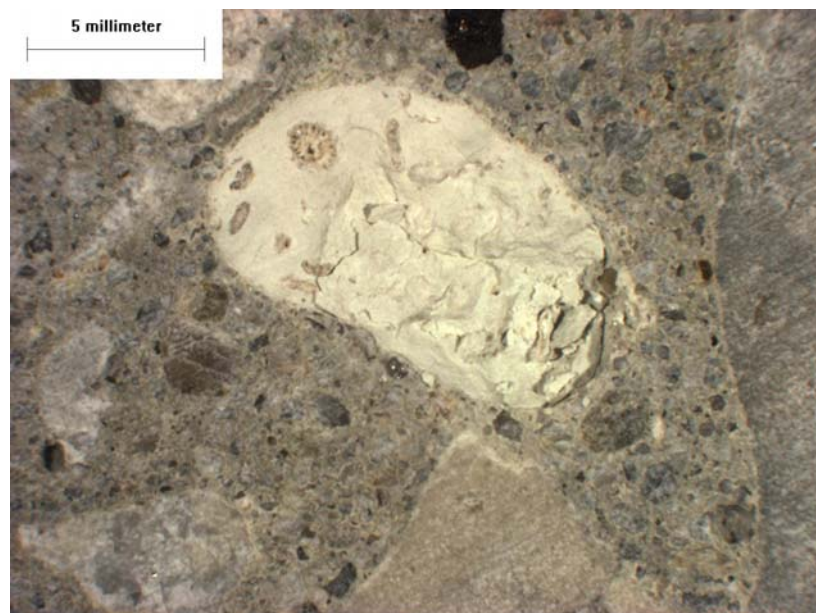


Figure 37. A 10-mm aggregate particle damaged after 19 cycles of freezing and thawing in water after exposure to potassium acetate solution.

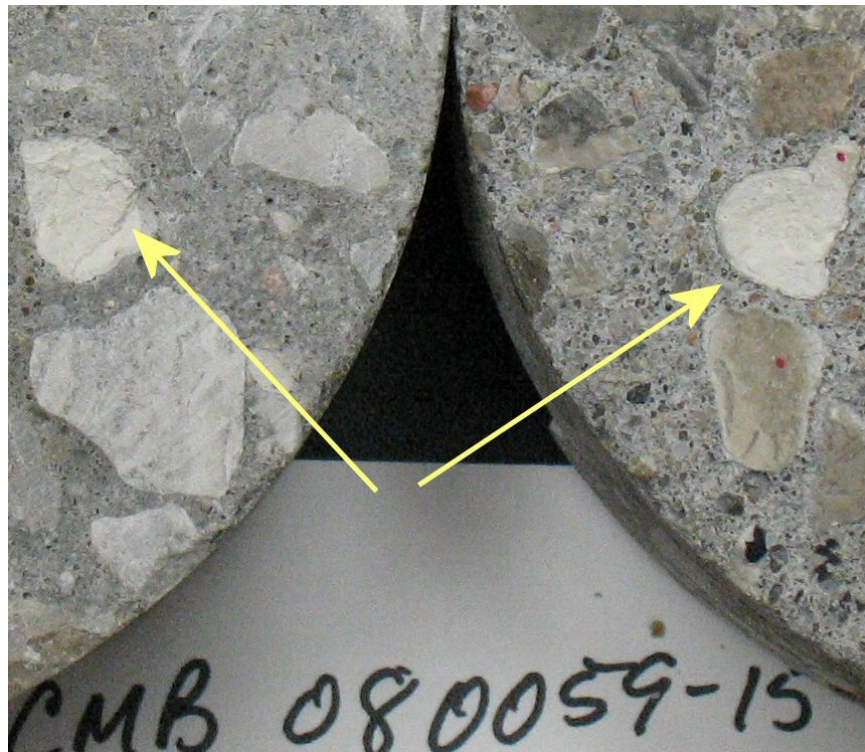


Figure 38. Comparison of control (left) and potassium acetate soaked (right) specimens from Core 15 after 19 cycles of freezing and thawing. The paste fraction of the specimen on the right is etched to about 1 mm. The yellow arrows show a type of coarse aggregate particle common in the Offutt pavement that deteriorates early and about equally with or without potassium acetate treatment.

Examination of thin sections showing the microstructure through the thickness of the specimens showed no evidence that microscopic damage had penetrated any deeper than the visible surface damage.

Figure 39 does show macroscopic damage in the form of a crack that apparently formed as a result of several nondurable aggregate particles located in proximity.

Figures 40–44 show the five samples in comparisons between the control- and the potassium acetate-exposed faces. These figures give some indication of the approximate abundance of affected aggregate.

The amount of damage to the paste fraction, as judged visually, appeared to be about the same for all potassium acetate-exposed specimens in this set. Air contents and spacing factors, which had been determined on different subsamples, ranged from 4 to 7.5%, and 0.21 to 0.42 mm, respectively. Depth of damage appeared to be unrelated to this variable.

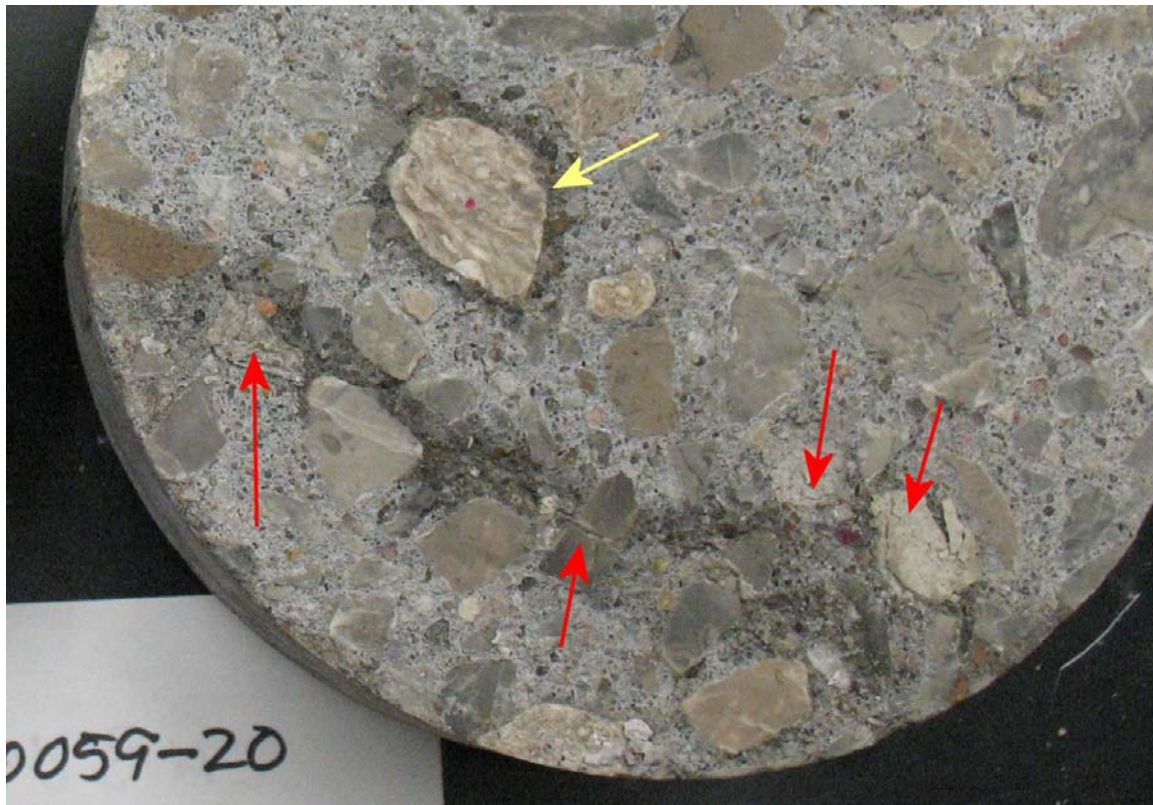


Figure 39. Sample from Core 20 soaked in potassium acetate, then exposed to 19 cycles of freezing and thawing. Red arrows indicate damaged aggregate that appear to have caused a crack (dark zone). The yellow arrow indicates a zone of paste around a coarse aggregate particle that appears to be preferentially taking up potassium acetate.

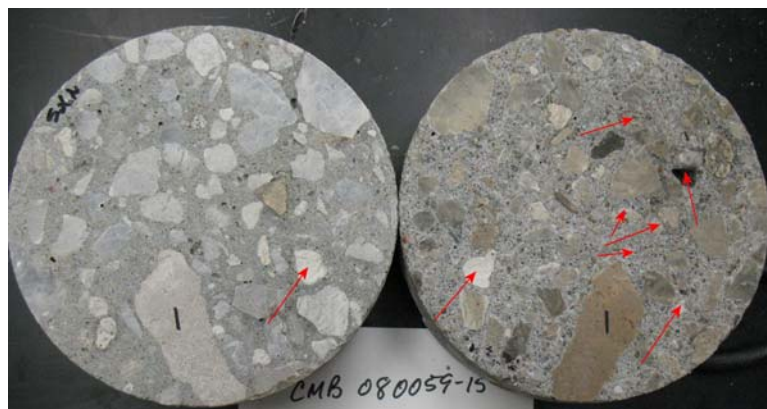


Figure 40. Sample from Core 15. Control is on the left, KAc-exposed is on the right. Red arrows indicate a damaged aggregate particle.



Figure 41. Sample from Core 16. Control is on the left, KAc-exposed is on the right. Red arrows indicate a damaged aggregate particle.

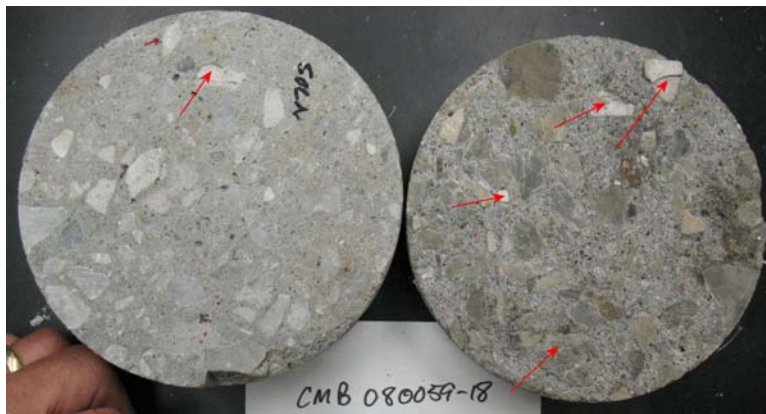


Figure 42. Sample from Core 18. Control is on the left, KAc-exposed is on the right. Red arrows indicate a damaged aggregate particle.

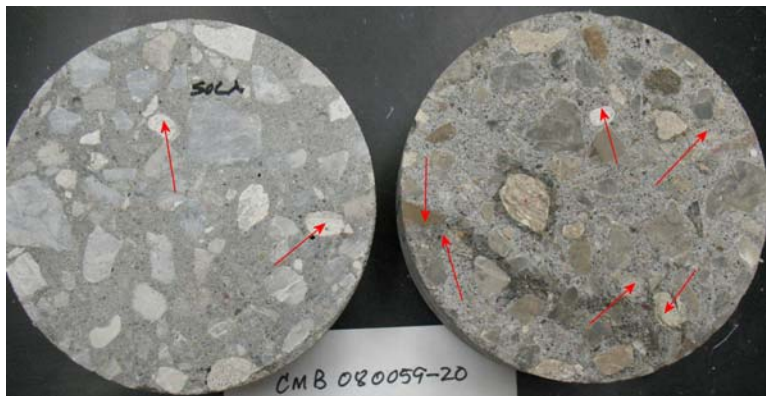


Figure 43. Sample from Core 20. Control is on the left, KAc-exposed is on the right. Red arrows indicate a damaged aggregate particle.

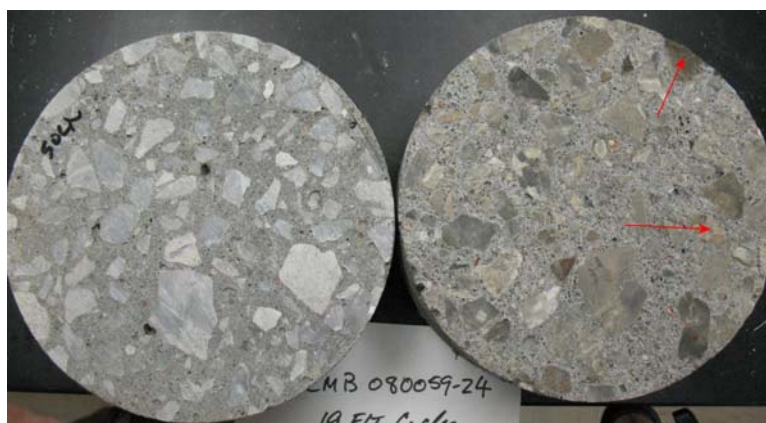


Figure 44. Sample from Core 24. Control is on the left, KAc-exposed is on the right. Red arrows indicate a damaged aggregate particle.

In other exploratory work on this test, it appeared that the degree of polish to the exposed surface imparted by laboratory preparation might be significantly related to the rate of paste damage. The amount of surface polishing would plausibly affect the penetration of salt solution into the concrete.

Three samples from Eppley Field were also tested for reaction to KAc exposure in the same way. Paste damage developed similarly in the Offutt samples. Aggregate damage was present, but appeared to involve fewer aggregate particles and developed later in the test period. However, it should be noted that sample sizes here are small, so definitive comparisons are not possible.

The Eppley samples were the same ones used for C 457 analysis. Air contents of these ranged from 4 to 6%, and spacing factors ranged from 0.20 to 0.25 mm. As in the Offutt samples, these showed no relationship to the apparent amount of damage.

Analysis of aggregate damaged by cycles of freezing and thawing

Mineralogical analysis of aggregate was not part of the original project plan. Once it became apparent that aggregate durability might be a more significant part of the damage process than originally thought, an investigation was planned to evaluate the mineralogy of the offending aggregate(s), to determine whether some unique features could be identified to anticipate poor performance.

As reported in Poole et al. (2007), it was not originally thought that the coarse aggregate was unsound with respect to freezing and thawing. This judgment was based on the absence of visible clay layers in the rock, which usually indicates the possibility of unsound aggregate. Subsequent to that report, popouts on Taxiway P and other places became evident (Figure 45). This alerted the researchers that nondurable coarse aggregate might be part of the joint-spalling and corner-cracking problem.

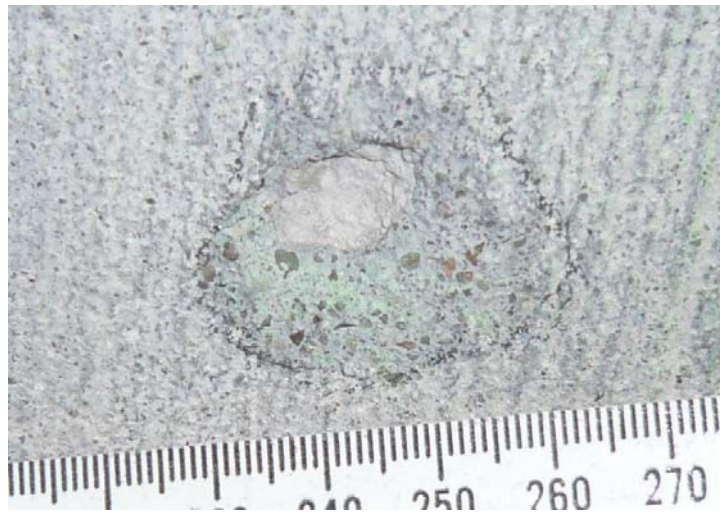


Figure 45. Surface popout on Taxiway P. From Core 21. About 10 mm is exposed, but on dissection this particle turned out to be about 20 mm in diameter.

An example of nondurable coarse aggregate apparently causing surface cracking in some slab corners on TW P is shown in Figures 46 and 47.

Figures 48 and 49 show a set of XRD patterns representing most of the coarse aggregate particle types found in the concrete. Calcite, the typical limestone mineral, was the principal mineral in most of the aggregates. Dolomite or ankerite (a modified dolomite mineral) was significant in others.

One type of aggregate particle appeared to be responsible for all of the observed popouts. This judgment was based on the unusual white color of the aggregate in all of the observed popouts.

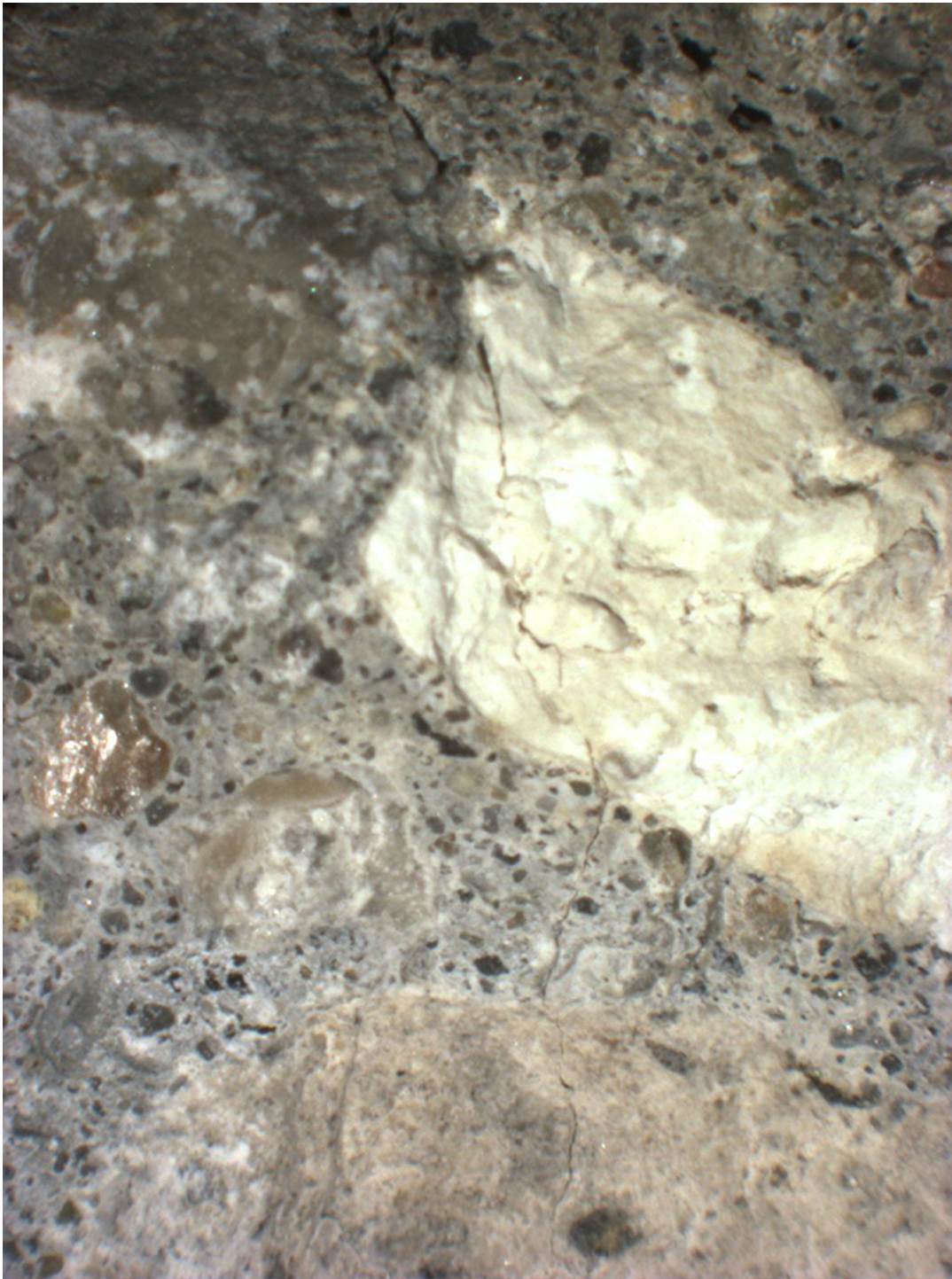


Figure 46. Vertical face of the crack shown in Figure 47. Two fragments of 20- to 25-mm coarse aggregate observed to be nondurable in laboratory testing, which appear to be involved in the origination of the crack. The finished surface is at the top edge of the picture.



Figure 47. View of a crack on the finished surface of Core 19, taken from TW P. The vertical face to the left is part of an intersecting crack. A piece of nondurable coarse aggregate is visible as the white feature on the vertical face.

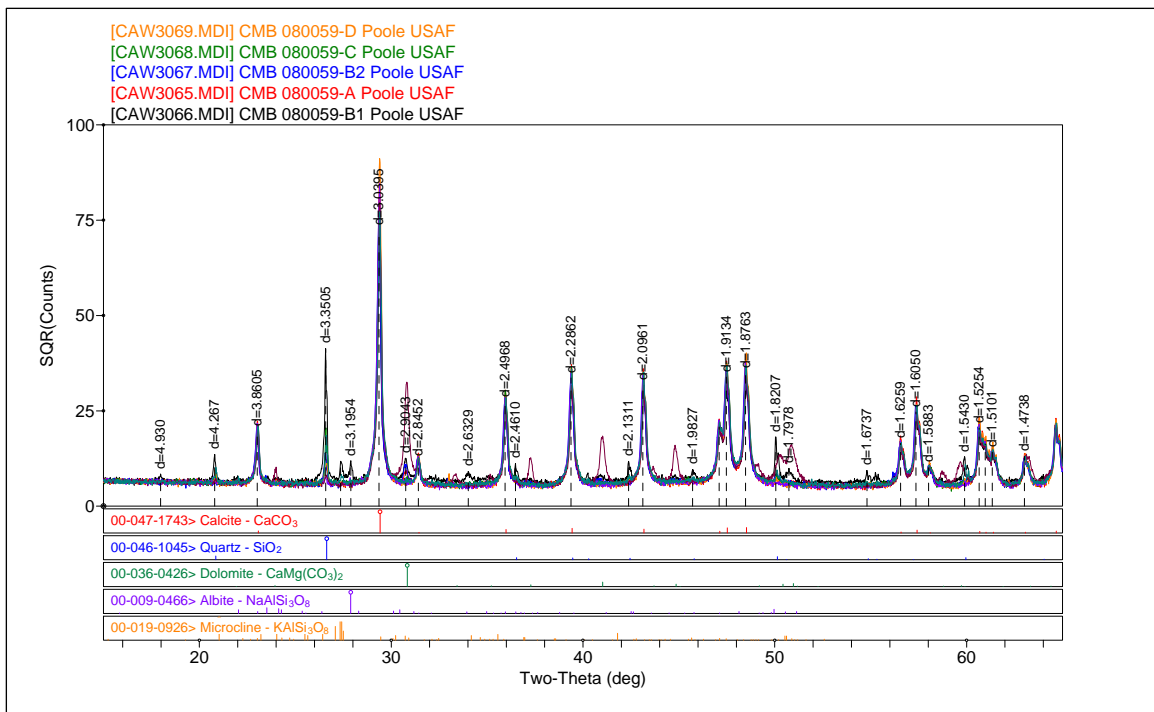


Figure 48. Common rock mineralogy found in the Offutt concrete.

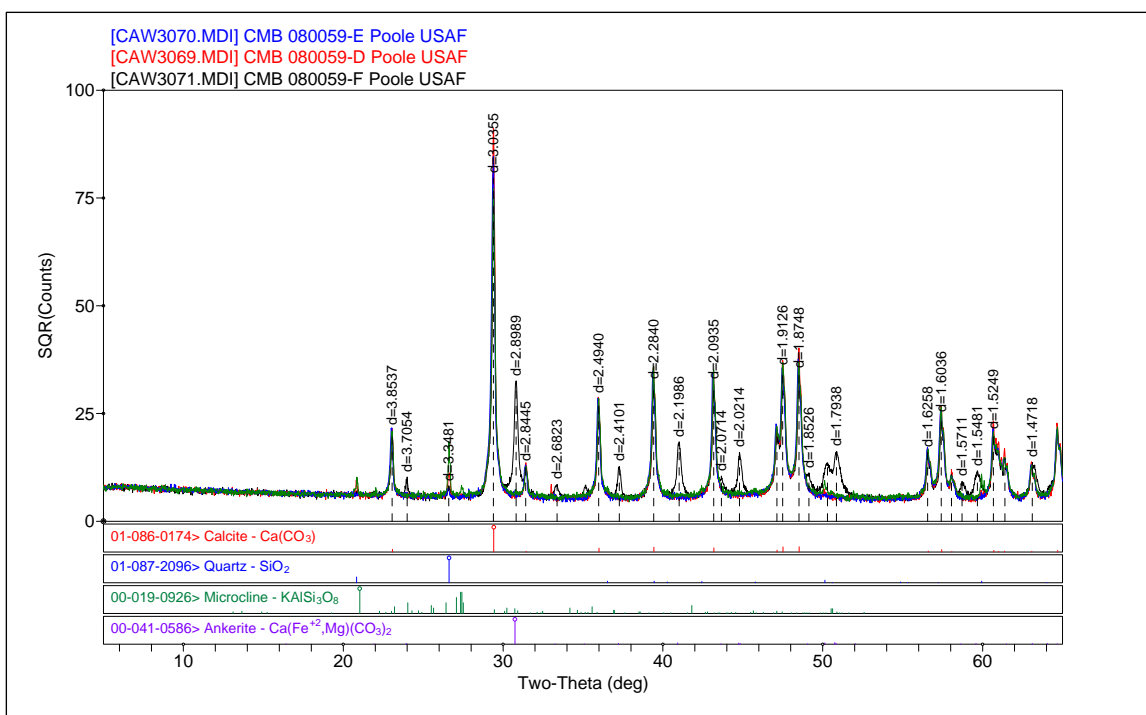


Figure 49. Common mineralogy of coarse aggregate found in Offutt concrete.

Figure 50 shows the XRD patterns for this rock type, which features ankerite more than most of the other mineral forms found in the concrete. This detail may be significant, since ankerite (in the literature occasionally and also in the authors' experience) is an indicator of rocks susceptible to alkali-silica reaction, and one reference was found for its apparent sensitivity to calcium chloride deicing salts (Dubberke and Marks 1987). There was also a small amount of the clay, muscovite, which could also contribute to unsound performance.

Figure 51 shows XRD patterns for three other damaged aggregate particle types found in C 666 specimens that appeared to be other than the white dolomite described above. Two of these (patterns P and M) appear to be the same mineralogy as the suspect dolomite described above. The other pattern is a pure limestone, but contains a small amount of gypsum, which could be significant, depending on the abundance of this aggregate in the concrete.

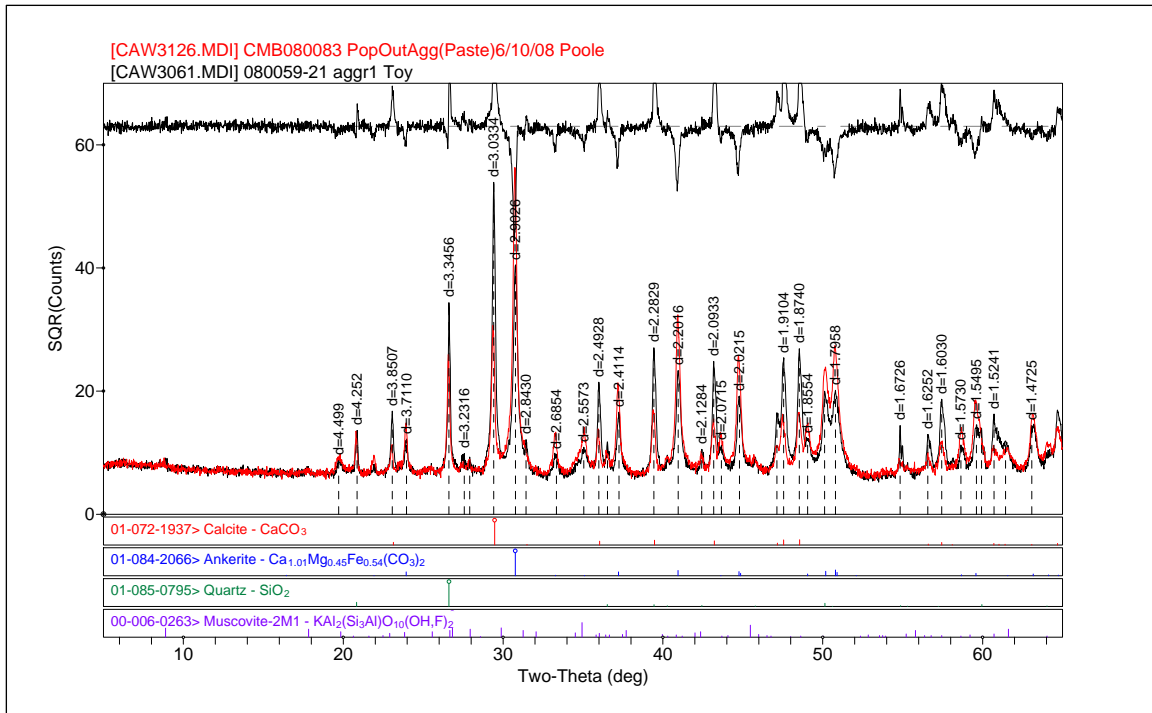


Figure 50. XRD patterns for coarse aggregate particles type causing the common surface popouts observed in the new construction. The rock is a dolomite with higher amounts of the mineral ankerite than the other aggregates used in the concrete. The pattern in red is a composite sample made up of rock from a number of popouts in Taxiway C. The pattern in black is the aggregate particle in Core 21 (Figure 45).

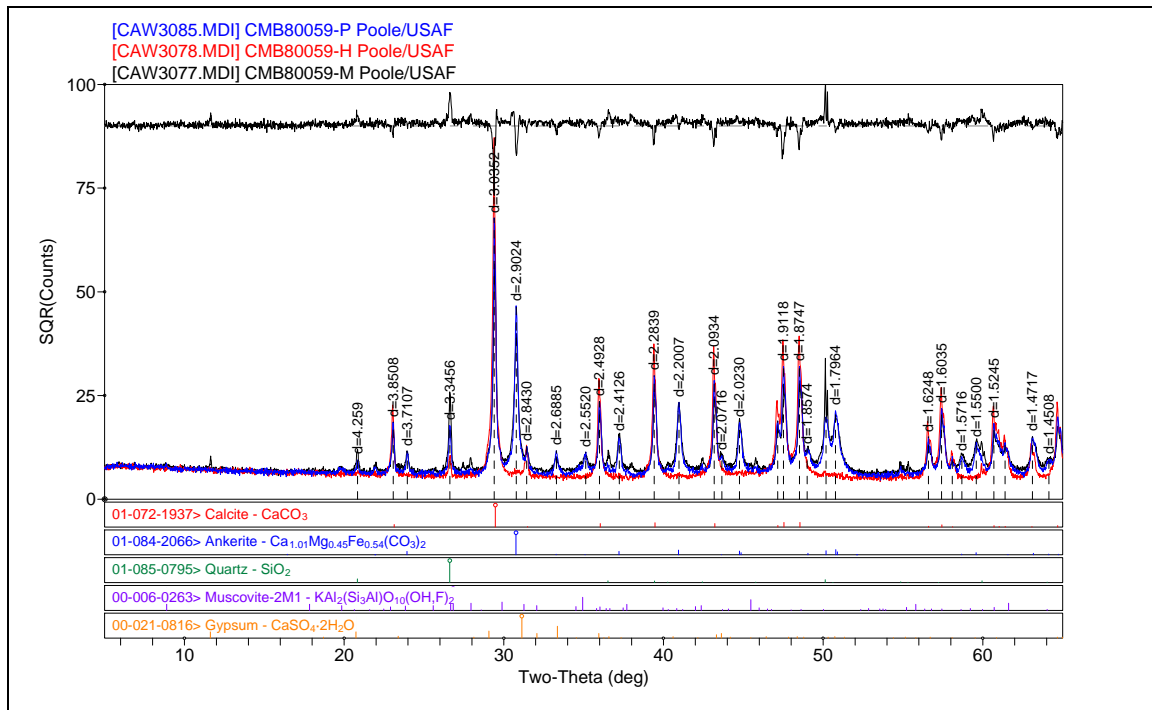


Figure 51. Other coarse aggregate mineralogies found in rock damaged by high numbers of cycles of freezing and thawing.

Air content of fresh concrete

The purpose of collecting and analyzing the air content on the fresh concrete was to verify compliance with project specifications. The analysis reported in Poole et al. (2007) found evidence of low air in hardened concrete, which is attributable to insufficient air in the fresh concrete, to losses of air during placing and consolidation, or a combination of the two.

The project specification for the air content of the concrete as delivered to the paving site was $6 \pm 1.5\%$. Tables 3 and 4 summarize measured air contents of fresh concrete as delivered to the paving site for slabs adjacent to the centerline of Taxiways P and C, and for the centerline slabs of the runway. The centerline was generally the most damaged part of these structures.

Table 3. Air contents (% by volume of concrete) of concrete Taxiway C and P centerlines as reported in QA data.

Placing Date	Lot No.	Air Content Tests, %							
08/27/06	35A&35B	5.3	5.1	5.5	4.6	5.0	4.6	4.8	4.8
09/14/06	46	5.1	5.0	5.3	4.8	5.1	5.2	5.0	5.1
09/22/06	50	4.6	5.0	5.1	5.7	4.7	5.1	5.0	5.7
09/26/06	52.A	5.3	4.6	4.7	4.7	5.3	5.7	5.8	5.3
10/03/06	57A	5.0	4.5	4.6	5.1	5.0	5.1	4.8	5.3
10/14/06	62B	5.7	5.1	5.7	5.3	5.2	5.3	5.0	5.0
Summary statistics, all data:	Average	5.1							
	Std dev	0.34							
	CV	7%							
	Kurtosis	-0.53							
	Skewness	0.07							

Table 4. Air contents (% by volume of concrete) of concrete in the Runway centerline as reported in QA data.

Placing Date	Lot No.	Air Content Tests, %							
7/03/06	5	4.5	4.6	4.5	4.6	5.8	4.8	6.2	5.5
7/14/06	10B	5.5	5.5	5.4	5.4	5.4	5.4	5.3	5.3
7/20/06	13	5.6	5.6	5.4	5.4	5.5	5.1	6.1	4.9
7/21/06	14	6.0	5.1	6.0	5.7	6.3	5.6	5.3	5.2
7/22/06	15	5.7	5.7	5.2	5.7				
7/30/06	19	5.1	6.1	5.6	5.9	6.2	6.2	5.8	5.9
7/31/06	20	6.4	6.2	5.5	6.0	5.7	5.8		

8/02/06	21	6.2	5.8	5.3	5.7	5.8	5.7	5.9	6.0
8/3/06	22	6.0	5.5	5.4	5.5	6.0	5.3	5.6	5.3
8/10/06	24	6.1	5.4	5.4	5.5	5.4	5.4	5.4	5.7
8/22/06	32A	4.8	5.8	5.5	5.0	4.6	4.9	4.6	5.0
9/5/06	38A & 38B	5.3	6.2	6.2	4.6	5.2	5.0	5.3	4.9
9/5/06	39A & 39B	6.1	5.8	5.3	4.7	5.3	5.2	5.3	5.1
9/6/06	40A - 40D	5.2	5.5	6.0	6.1	6.0	6.1	5.3	5.3
9/7/06	41C	6.8	5.3	6.1	6.0	5.2	5.3	5.8	5.3
9/11/06	43E	5.3	5.2	5.4	5.3	5.1	5.5	5.3	5.0
10/3/06	57B & 57C	5.0	4.5	4.6	5.1	5.0	5.1	4.8	5.3
10/4/06	58C & 58D	5.2	5.0	5.1	5.3	4.8	4.8	5.2	5.0
10/6/06	60B - 60D	5.0	5.7	5.0	5.1	5.1	5.4	5.0	5.4
10/14/06	62D & 62E	5.7	5.1	5.7	5.3	5.2	5.3	5.0	5.0
10/16/06	63A-63C& 3E	5.0	5.8	5.8	5.9	5.0	4.7	5.9	5.3
10/17/06	64A - 64H	4.6	5.0	5.0	5.1	5.3	5.0	5.3	5.0
Summary statistics, all data:	Average	5.4							
	Std dev	0.45							
	CV	9%							
	Kurtosis	0.35							
	Skewness	0.15							

The air content was very well controlled at levels between 0.5 and 1.5%, lower than the center of the target range (6%). The standard deviation was 0.34% for the Taxiway P and C concrete and 0.45% for the runway concrete. The distribution of test results appears to conform to a purely random pattern, with no significant evidence of skewness or kurtosis. The standard deviation of the test method, as reported in ASTM C 231, is 0.28%. Therefore, it appears that the control of the actual air in the concrete was very good. No tests exceeded the limits of the required range.

Properties of entrained air of hardened concrete

The purpose of this analysis was to verify the analysis in Poole et al. (2007) with more samples and to use the data to try to link air void properties with the laboratory measure of durability (C 666). The 2007 analysis found some evidence of parameters being outside recommended limits, but concluded that this was not plausibly the cause of the damage during the first winter. There was disagreement on this point.

Table 5 summarizes air content, spacing factor, and specific surface for all standard C 457 determinations on hardened samples.

Table 5. Properties of entrained air from hardened concrete samples of concrete from Offutt AFB pavement.

Sample Number	Air Content, %	Spacing Factor, mm	Specific Surface, mm ⁻¹
1f	1.9	0.26	27
3	2.8	0.19	30
10	2.3	0.18	37
11p	2.8	0.26	24
12f	3.7	0.25	22
14	1.5	0.31	25
15	4.6	0.25	18
16	7.5	0.24	13
18	5.4	0.42	11
19	2.6	0.21	17
20	4.0	0.22	20
21	5.9	0.21	18
22-1	4.2	0.24	19
22-2	3.8	0.20	22
23-1	4.4	0.29	14
23-2	4.5	0.18	27
24	4.2	0.21	22
26-1	6.7	0.23	15
26-2	4.0	0.25	21
27-1	1.4	0.28	25
27-2	3.4	0.23	24
28	4.6	0.31	12
Mean	3.9	0.25	21
Std	1.57	0.053	6.3
CV	40%	22%	30%
Kurtosis	0.16	4.26	0.51
Skewness	0.43	1.65	0.48

Note: C 457 P&B (precision and bias) statement:

Air content, standard deviation = 0.73.

Spacing factor, CV (coefficient of variation) = 7%.

Specific surface - no value given.

There were some replicates using different subsamples from the same core. The simple descriptive statistics seem to contain some interesting and potentially useful information.

A notable detail of the data presented in Table 5 is the air void properties of Core 18. The air voids in this concrete had a very large spacing factor and low specific surface. This core was taken from a runway slab (south side of the centerline) very close to the parallel joint with the shoulder slab. This joint exhibited a strong damage pattern that appeared to be different from the sliver spalling seen elsewhere in that it showed a more traditional freezing-thawing damage pattern (Figure 52). The damage was in the form of very thin spalls relative to what was seen in the damage on Taxiways P and C. This zone is where Core 13 from the spring 2007 sampling was taken on the south side expansion joint at Station 39+44. It appears likely that the concrete in this vicinity is seriously deficient in properly entrained air and that this is likely to be a major contributor to the damage.



Figure 52. Expansion joint on south side of RW centerline at STA 39+44. Runway slab is to the top.

As indicated by the mean air content reported in Tables 3–5, the total air of the fresh concrete was significantly higher than in the hardened concrete. On the average, there was a loss of air of 1.4%. Probably, the significant question is whether this loss is functionally significant. It is common knowledge that some air is often lost during placement and consolidation without particular detriment to the spacing factor and specific surface,

commonly thought to be the most significant parameters of the entrained air system (ACI 1998b).

Another significant comparison between the fresh and hardened concrete is the variability in air content. The coefficient of variation (CV) of the fresh concrete was found to be about 8%, while the CV of the hardened concrete was found to be 40%. The extremes in air contents in the hardened concrete were from 1.4 to 7.5%. Some samples contained more air than was measured in the fresh concrete. Even given the error in air content determination ($s = 0.73$, as given in the C 457 precision and bias statement), the variability in the total-air data suggests that significant redistribution as well as loss of air in the hardened concrete may be occurring.

The spacing factor and specific surface properties are generally considered to be more important in the context of durability to cycles of freezing and thawing than is total air. Of the two, the spacing factor is generally given more attention than the specific surface. The mean of these properties is not particularly notable. The spacing factor is larger than the ACI recommendation of 0.20 mm but, in the context of the sample standard deviation, this difference is not statistically significant. The average specific surface of 21 mm^{-1} is in line with recommendations.

Perhaps the most notable feature of the descriptive statistics is the variability and skew in the spacing-factor data. Figure 53 shows the frequency distribution in this property. The skewness statistic of 1.65 is greater than the critical value of 1.28, which is the maximum expected for a data set of this size if values are purely random. There were a number of observations of spacing factor around 0.30 mm and higher (maximum observed = 0.42 mm). A spacing factor of 0.30 mm may be the upper tolerable value for concrete experiencing many cycles of freezing and thawing while saturated.

This variability in spacing factor is well outside that expected from the random error of the test method, 7% (CV) reported in C 457. The CV among the data reported in Table 5 is 20%. This indicates that the observed high values are not plausibly attributable to observations simply being on the tails of a normal random frequency distribution.

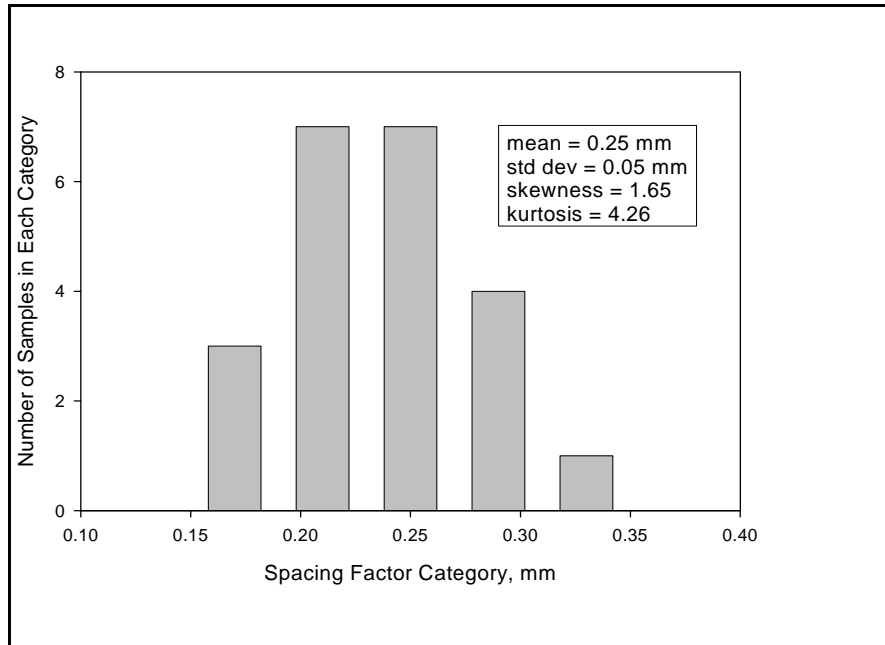


Figure 53. Frequency distribution of spacing factor among samples.

Table 6 presents repeatability of air void parameters among cores taken from the same placement of concrete, usually within about 1 meter. This gives a measure of variability in the concrete on approximately a 1-m scale.

Table 6. Repeatability of air void parameters in cores taken in close proximity within the same placement of concrete.

Core ID (Sorted by Concrete Placement)	Percent Air	Spacing Factor mm	Specific Surface mm-1
1F	2.8	0.19	30
27	2.4	0.25	24
23	4.5	0.24	21
24	4.2	0.21	22
3	1.9	0.26	27
10	2.3	0.18	37
22	4.0	0.23	21
12F	3.7	0.25	22
16	7.5	0.24	18
11P	2.8	0.26	24
15	4.6	0.25	11
Std dev within placement	1.55%	0.027 mm	6.6%
CV	42%	12%	28%

How well a single determination of air void properties represents an entire lot of concrete may be a significant issue. In practice, the properties of concrete determined by a single C 457 determination are generally taken to represent the entire lot of concrete placed. In this case, a lot is reasonably taken to be a single delivery of concrete to the paving site, although the concept is probably a little more complicated. This is typically several cubic meters. This assumption would be valid if it can be assumed that the concrete is truly uniform throughout the lot.

Probably an important point is that the criticality of damage due to freezing and thawing is appropriately characterized as a local, small-scale event rather than an average event taken over several cubic meters. Therefore, small-scale variation in this property at the local level may be functionally significant. A single C 457 determination is normally executed over a prepared surface of about 100 to 200 cm². This represents a very small sample relative to the typical lot size of several cubic meters. Stated another way, the lack of durability of a section of concrete of only a few cubic centimeters may be a critical issue, depending on the location of the this small section of the concrete in the structure.

To further investigate this local variation issue, C 457 point counts were done on very small samples of concrete from thin sections made of analysis of damage in the C 666 tests. The location from which these thin sections were taken was not random. Some were taken to intentionally represent variable zones of damage in the test specimens. Some were taken from pieces of concrete sawed off of the core to fabricate the C 666 specimens, which were not exposed to cycles of freezing and thawing. The resulting data are summarized in Table 7.

Table 7. ASTM C 457 data representing small sections of concrete.

Sample	Specimen Size, cm ²	Total Air, %	Spacing Factor, mm	Specific Surface, mm ⁻¹	Notes
TS 27-1	7	1.8	0.25	32	Minor damage
TS 27-2	7	2.5	0.28	26	Close to damage
TS 27-3	7	6.3	0.25	19	Unexposed to F/T
TS 18-1	7	3.8	0.47	12	Modest damage
TS 18-3	7	12.1	0.51	6	Major damage
TS 18-4	7	4.5	0.49	10	Unexposed to F/T
TS 18-5	7	5.2	0.29	16	Unexposed to F/T

TS 24-1	7	5.4	0.39	11	Minor damage
TS 24-2	7	5.6	0.52	9	Minor damage
TS 15G	15	4.7	0.58	9	Major damage
TS 15B	15	4.3	0.38	13	Major damage
23G	25	3.8	0.16	33	Modest exterior damage
23B	25	8.9	0.17	17	Modest exterior damage
Mean		5.2	0.36	16	
Std dev		2.96	0.14	8.8	

Strictly speaking, analysis of such a small-specimen surface area does not comply with procedures in C 457, in that a smaller number of points are included in the analysis than recommended. This would result in an increase in the random error of the method for determination of air void properties by an amount estimated to be about 2.5-fold. Based on the precision reported in C 457 (see footnote to Table 5), this would result in a random error of about 20% (CV).

Even allowing for the inflated error of the method resulting from this modification, these data do support the concept that there are relatively small zones of concrete that may have very poor air void properties. In two cases, the measured spacing factor was notably better than average. When buried deep in a mass of concrete, loss of strength by these small volumes of poorly air-entrained concrete may not be too critical. However, if these exist close to joints, concrete surfaces, or nondurable coarse aggregate particles, they could be expected to result in visible, and maybe significant, damage. The extent of this analysis is not large enough to give a good estimate of the frequency of such poor-air zones of the concrete structure under study, so no large-scale estimate of the importance of this finding can be made at this point.

Table 8 summarizes air void parameters of the three samples from Eppley Field and Mountain Home AFB. Sample numbers were too low to make substantial statistical comparison to the Offutt concrete. The results indicate average properties similar to the Offutt concrete. There was not enough data to analyze for the variability and skewness, as in the analysis of the Offutt concrete.

Table 8. Properties of entrained air from samples taken Eppley Field and Mountain Home AFB.

Sample Number	Air Content, %	Spacing Factor, mm	Specific Surface, mm ⁻¹
<i>Eppley Field</i>			
3	5.0	0.21	21
4	5.6	0.20	23
7	3.8	0.25	21
Mean	4.8	0.22	22
Std	0.92	0.026	1.1
CV	19%	12%	5%
<i>Mountain Home AFB</i>			
1	6.0	0.18	20
2	6.7	0.32	11
3	5.8	0.23	19
Mean	6.2	0.24	17
Std	0.47	0.07	4.5
CV	8%	29%	26%

Monitoring during winter 2007–2008

Monitoring of the pavement damage during the winter of 2007–2008 was planned as an auxiliary investigation to help refine thinking on the cause of the damage. It was thought that one set of possible causes should continue to be active after the first winter, while others predict a slowing down.

The two areas chosen for monitoring development of damage during the winter of 2007–2008 both had considerable damage to the centerline sections in the form of joint spalls during the winter of 2006–2007, but only moderate damage to the first set of longitudinal joints parallel to the centerline and the transverse joints. These monitoring sites are documented in Chapter 2.

Damage maps were obtained from the Base civil engineering office. These showed the damage as of Dec 07 along the centerline to be nearly continuous in some places, but also showed some sections having only isolated damage. Some parts of the centerline had essentially no damage. It was

not possible to estimate accurately the number of damage events along the centerline because of the almost continuous overlapping form of damage. However, the number was certainly more than 20 events per 20-ft segment in the sections of joint that were damaged, or more than 200 over the 12-slab length of centerline monitored. The off-centerline joints contained about 1 to 2 damage events per 20-ft section, or about 30–50 damage events over the 30-slab (24 longitudinal + 6 transverse joint) length of joint monitored. The transverse joints and the off-centerline joints appeared to have little difference in rate of damage.

Damage that was noted as having occurred over winter 2007–2008 appeared to fall into four categories:

- **Surface abrasion.** This was a minor form of damage usually noticed where joints were painted (centerlines) because of the scraping away of the paint. Figure 54 shows an example of this.



Figure 54. Surface abrasion along a centerline joint.

- **Loss of surface paste.** This is another minor form of damage. In these cases, some of the texture of the surface paste was lost and subsurface coarse aggregate particles exposed. Figure 55 shows an example of this.



Figure 55. Loss of surface paste and formation of a new spall. Condition in December 2007 is on the left, and condition in June 2008 is on the right.

- **Raveling of existing damage.** This is a more substantial form of damage that occurred only in zones of the joint damaged in the first winter. This damage appeared as loss of material along the relatively sharp edge where sliver spalls were separated from the finished surface of the concrete. Figure 56 shows an example of this.

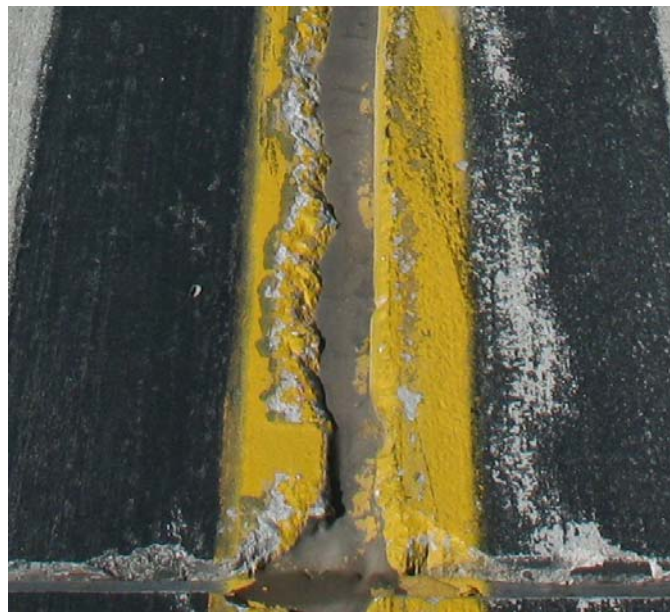


Figure 56. Raveling in the winter of 2007–2008 in damage first created in the winter of 2006–2007.

- **New sliver spalls.** These are spalls that resemble the large number of spalls that developed during the first winter after construction, as in Figure 11.

In general, the damage over the winter of 2007-2008 was light relative to the damage that had developed over the first winter. Table 9 summarizes the observed total number of the four categories of damage events.

Table 9. Number of observations of new damage events on or near joints.

Type of Damage	Taxiway P	Taxiway C	Notes
Abrasion	6	3	Randomly distributed between CL and off-CL joints.
Loss of surface paste	3	9	Randomly distributed between CL and off-CL joints.
Raveling	9	10	Predominantly a CL event in zones of old damage.
New sliver spalls	3	10	Randomly distributed between CL and off-CL joints, about half in old-damage zones in CL.

Several of the damage events were picked up in the monitoring, suggesting potentially significant things about the damage process. None of these was noted while actually on the pavement, but were noted later during examination of the photographs. Consequently, some of the observations must be classified as tentative because of some missing details that would be revealed with closer inspection.

The noted phenomenon of loss of surface paste in some zones, accompanied by exposure of coarse aggregate, may have implications for the observed pattern of joint spalls (as shown in Figure 55). The aggregate exposed appears to be the nearly white coarse aggregate—determined in the freezing and thawing tests, the potassium acetate tests, and the surface popouts in the structure as being particularly susceptible to damage from cycles of freezing and thawing. The loss of paste may be a result of surface damage from these underlying aggregate particles or from potassium acetate treatment, or both. More significantly though, this damage may reveal the pattern of distribution of this susceptible aggregate as being in random clusters. Where a cluster intersects a joint, there is the potential for this aggregate to contribute to joint damage.

Some of this white nondurable aggregate also appears to be showing up in the zones where significant raveling occurred during the winter. Figure 57 shows an example of one of these zones.



Figure 57. Aggregate suspected of being nondurable in a zone of heavy raveling.

During the Dec 07 monitoring visit, it was noted that the surface concrete near a number of the joints had retained a wet look, apparently from a rain 4 days earlier. It seemed unusual for the wet discoloration to persist for this long, although this kind of thing is common when certain deicing salts are used because of the hygroscopic nature of these materials.

Figure 58 shows a comparison of a joint segment in Dec 07 and Jun 08, which shows the joint's wet appearance in December and the apparent loss of surface paste by June. This suggests that the potassium acetate used on these structures may be having some effect on the pavement surface where it tends to reside after applications.

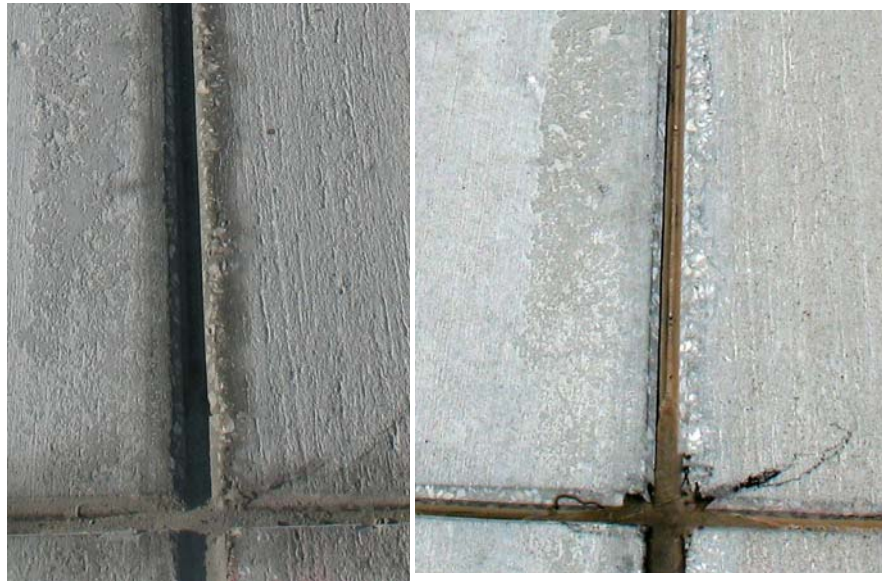


Figure 58. Possible case of deicing salt effect on surface paste. Wet joint in Dec 07 is on the left. Dry joint in Jun 08 is on the right.

Service loads are one of the factors considered significant as a secondary mechanism in revealing the damage to the concrete by weathering-related mechanisms. Figures 59 and 60 show examples of loads applied probably by a snowplow blade and by aircraft-wheel loading, respectively.



Figure 59. Scars that appear to result from snowplow abrasion, along with damage to the joint (lower right).

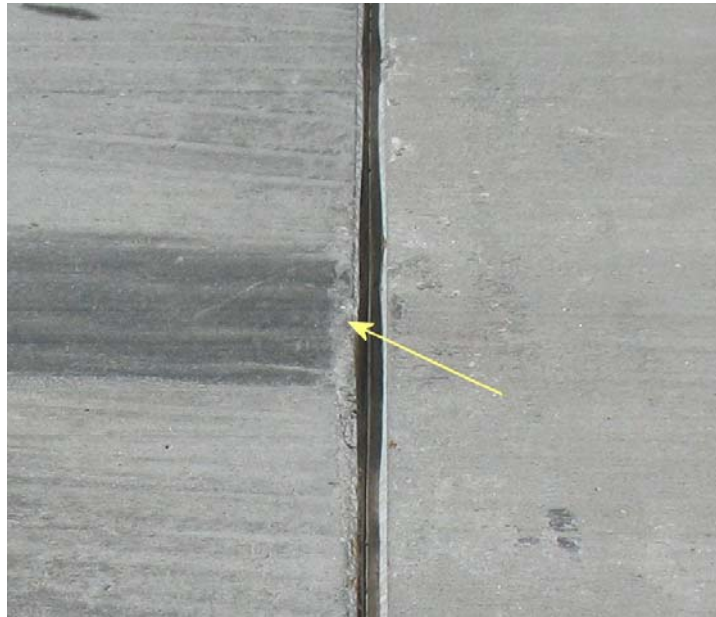


Figure 60. Tire damage along a transverse joint.

A final observation from the monitoring photographs is that there seems to be a definite pattern of large popouts caused by ~25-mm coarse aggregate on TW P relative to TW C. It is well known that gross damage to pavements in the form of D-cracking tends to be associated with larger sizes of frost-susceptible coarse aggregate. That the incidence of cracking in corners is higher on TW P than on TW C is plausibly related to this apparent size effect. Figures 46 and 47 show a case of this in a TW P site.

Analysis of maturity effects

The purpose of this analysis was to investigate the possibility that cold weather during the late-season placing of some of the concrete might be part of the damage scenario. Two processes—freezing events early in the concrete soon after placing, and low strengths when service loads are applied due to retarded hydration—are possible damage features of cold-weather concreting. The observed damage would be consistent with either or both of these processes.

The concrete in Taxiways C and P was placed in fall 2006. Almost all of the concrete were placed on TW C by 15 Oct 2006. Placement of several individual slabs along the runway centerline was made on 17 Oct. Concrete was placed in another unidentified location on 19 Oct 2006. Service loads were allowed on 6 Nov—22 days after most of the final concrete was placed and 18 days after the last placement.

The major placements of concrete along the centerline of TW P were made on 14 and 22 Sep; along the centerline of TW C on 26 Sep and 3 Oct; and along the runway centerline on 6 Sep (see Tables 3 and 4).

Weather is cooler in Omaha at this time of the year, so strength gain of the concrete is expected to have been somewhat slower. Thus, there is at least reasonable question about the strength of the concrete when service loads were first applied. There was also some freezing weather within a few days after the last concrete was placed. Early freezing of green concrete is known to cause damage.

The objective of this analysis is to apply the maturity concept (ASTM C 1074) to the concrete strength gain in order to estimate the equivalent age of the concrete when the first freezing events occurred and when service loads were applied. The maturity equation in C 1074 was used to estimate equivalent ages near the time of freezing and loading events for the two latest placements (worst-case scenario).

$$t_e = \sum e^{-Q\left(\frac{1}{T_a} - \frac{1}{T_s}\right)} \Delta t \quad (1)$$

where:

- t_e = equivalent age (in days) at temperature T_s (units of K)
- Q = activation energy, in this case taken to be 5,000 K
- T_a = average temperature of the concrete during time interval Δt (K)
- T_s = standard laboratory curing temperature (K) = 296 K
- Δt = time period under consideration, in this case, 1 day

Unconfined compressive strength (UCS) data used to define a strength gain curve at 296 K are shown in Table 10.

Table 10. Laboratory data at 296 K used to develop a strength-age or equivalent-age curve.

Age, days	Log ₁₀ Age	UCS, psi
7	0.845	3,910
14	1.146	4,500
56	1.748	6,100
Equation used to calculate UCS for a give equivalent age (t_e) at temperature T_a :		
Strength, psi = 1773 + log ₁₀ age(days)*2458		

Expected compressive strengths were then calculated using the strength data developed during the mixture verification procedure and distributed by Thiele Geotech in May 2006 for the 0.44 w/c mix (Table 11). Other mixes included 0.37 and 0.40. Strengths for all three mixtures over the 7-, 14-, and 56-day test ages reported were approximately the same. The curing temperature for these tests was assumed to be 23 °C, which is the standard curing temperature for laboratory testing.

Daily weather data for October and November 2006, for Lincoln, NE, were located from the National Weather Service Web site. It was assumed that these reasonably represented Omaha, approximately 50 miles away.

As a worst-case analysis, the maturity of concrete placed on 17 and 19 Oct was calculated and compared with dates of first significant freezing and opening to traffic. Table 11 shows the weather data for the mid-October/early November time frame. Maturity calculations are also presented for concretes placed on 17 Oct and 19 Oct 2006.

Three strength-development events are critical to this analysis:

1. Concrete should not be exposed to a single freezing event, if saturated, until compressive strengths of at least 500 psi are reached.
2. Concrete should not be exposed to cycles of freezing and thawing, if saturated, until compressive strengths reach at least 3500 psi.
3. Concrete compressive strengths must be sufficient to sustain service loads before being put into service. We do not know what these strength requirements are, but according to the Thiele report, a compressive strength of 4,000 psi equates to a flexural strength of 650 psi.

As noted above, the concrete placements that have suffered the worst damage were along centerlines. According to the placing dates reported in Tables 3 and 4, these were placed at least 2 weeks earlier than represented by the above analysis. This concrete had more than 2 weeks of curing before the first freezing event and 34 days of curing before opening to traffic. It appears unlikely that maturity was an issue with the damage under analysis in this report.

Table 11. Weather data for Lincoln, NE, and maturity calculations for mid-October through early November 2006.

Weather Parameter						Maturity Calculations for Concrete Placed 17 Oct 06		Maturity Calculations for Concrete Placed 19 Oct 06	
Date	Avg Temp °C	Avg Temp °F	Freeze Event °F	Rain in.	Daily Eq Age	Cum Eq Age	Estim UCS	Cum Eq Age	Estim UCS
15-Oct	12	54		0.07	0.53				
16-Oct	14	57		0.01	0.58				
17-Oct	11	51			0.48				
18-Oct	3	37		0.14	0.29	0.29	452		
19-Oct	4	40	23		0.32	0.61	1251		
20-Oct	9	49		0.10	0.44	1.06	1833	0.44	908
21-Oct	5	41		0.11	0.33	1.39	2127	0.78	1507
22-Oct	4	39			0.31	1.70	2342	1.09	1866
23-Oct	3	37	23		0.29	1.99	2510	1.38	2118
24-Oct	4	40	23		0.32	2.32	2670	1.70	2342
25-Oct	4	40		0.26	0.32	2.64	2809	2.03	2527
26-Oct	8	46		0.07	0.40	3.04	2960	2.43	2720
27-Oct	12	54			0.53	3.57	3131	2.96	2930
28-Oct	10	50			0.46	4.03	3261	3.42	3084
29-Oct	13	55			0.55	4.58	3396	3.96	3243
30-Oct	11	51			0.48	5.05	3502	4.44	3364
31-Oct	-1	30	20		0.22	5.28	3548	4.66	3416
1-Nov	1	33	19		0.25	5.53	3598	4.91	3472
2-Nov	0	32	16		0.24	5.77	3643	5.15	3523
3-Nov	3	37	17		0.29	6.06	3696	5.44	3582
4-Nov	7	45			0.39	6.44	3762	5.83	3655
5-Nov	8	47	26		0.41	6.86	3828	6.24	3728
6-Nov	11	52			0.49	7.35	3902	6.74	3809

Note: Yellow highlight indicates significant freezing event; brown highlight indicates data of first service loads.

4 Discussion of Results and Integration of Information

Testing for durability to cycles of freezing and thawing

Forster (2006) is a good review of the test methods on this topic. ASTM C 666 is the standard test for testing the adequacy of air entraining in concrete, and is also the basis for ASTM Practice C 1646 for testing aggregate durability to cycles of freezing and thawing. It is recognized as being quite harsh, and the scope of the method cautions about trying to make real-time interpretations of the results. Also, as noted in Chapter 3, there was an indication that some of the damage to the specimens in the test may be artifacts of the method and not particularly relevant to field concrete.

The early failure of a number of the cores in the C 666 testing may have had a significant contribution from fractures that developed among some zones of relatively large, closely spaced voids filling with water during the test and then freezing in a confined space. The resulting damage appeared to propagate among these voids in the form of a single macrocrack. It is at least a debatable point that this form of damage does not happen with field concrete in a well-drained location.

However, in spite of this potential artifact, the authors do consider the method to reveal useful things about the durability of the cement paste and aggregate in the concrete. The damage to the cement paste from inadequate air entraining, and to concrete as a whole when aggregate is not durable, is normally detected in the C 666 test when the test is relatively far along. However, it is reasonable to assume that the damage actually starts on a microscopic scale much earlier than when detected using the test method. So, even though harsher than representative of the first winter's exposure to cycles of freezing and thawing, the test probably is useful for revealing nondurable materials in the concrete—information that plausibly has meaning in the analysis of early damage, although this cannot be verified directly with this test. Some indirect evidence based on field observations will help discriminate some of the results, as discussed below.

Two types of observations support the C 666 results. One type of observation that supports the nondurable aggregate hypothesis is that a particular type of dolomite aggregate (as described in Chapter 3) started to show damage in the field after the first winter, as indicated by the presence of popouts. Other aggregate types also showed unsound performance in the D 5312 testing (a much milder test method) when treated with potassium acetate. Another type of observation that supports the hypothesis that air void structure was inadequate in some places was the form of damage at a joint near where Core 18 was taken. The damage at the joint strongly resembled the typical laminated fracture appearance of typical damage from freezing and thawing when air void structure is inadequate.

Concrete cores from Eppley Field representing concrete placed at about the same time using approximately the same concrete mixtures also failed at about the same rate as the Offutt cores. The Eppley cores also appeared to suffer the same large void-related problem mentioned above, which is thought to be an artifact of the test. The Eppley concrete differed from the Offutt concrete in the source of coarse aggregate. The damage to coarse aggregate in the Eppley cores did not appear to be as frequent as in the Offutt samples.

ASTM D 5312 is a test method designed to test riprap for durability to cycles of freezing and thawing. The method was used in this work because the configuration of the test method was better suited to evaluating effects of deicing salts. The test method is less harsh than C 666 because of the rate of temperature cycling and the use of air as the temperature transmission medium instead of a water-antifreeze mixture. The exposure to the potassium acetate solution was somewhat realistic with respect to field practice in that exposure was a few days in length and not continuous. As used for this purpose, the test method gives nonquantitative results. However, the qualitative results were very revealing of early-age effects to both paste and aggregate, even without quantitative results.

The paste in the control samples showed no damage, while damage in the deicer-treated samples showed notable damage to a depth of about 1 mm. The test also revealed that only one aggregate type deteriorated at the early age represented in this test, in both the control and deicer exposure. This was the same white dolomite observed to be underlying popouts in the field concrete. The method did not reveal the irregularities in air entraining suggested by the C 666 results and the C 457 analysis. This may be

because the exposure time was not long enough (19 cycles executed). Also, the amount of sampling and testing using this method was not as extensive as in the C 666 and C 457 testing, so the probability of encountering zones of susceptible concrete was lower.

Damage patterns in the C 666 specimens were identified as deriving from both paste and aggregate damage due to exposure to cycles of freezing and thawing. This was based on visible observation of microscopic damage to both the aggregate particles and the cement paste, which is characteristic of the freezing-and-thawing damage mechanisms.

Microscopic damage seen in samples from in-place concrete exposed during the winter of 2006-2007 showed certain similarities to the laboratory-exposed samples. This strengthened the contention that the results of the C 666 testing had value in the present analysis, even though this testing was arguably more severe than the concrete experienced in service.

Nondurable coarse aggregate

The guide specification for heavy-duty pavement (UFGS 2006) directs that aggregates be tested for durability to cycles of freezing and thawing according to ASTM C 1646 only when the service record is not known. In the case of the Offutt concrete, the Weeping Water aggregate source had been used without problem for many years. In addition, the source was tested relatively recently prior to construction and was determined not to be a problem. Therefore, the standard aggregate tests for durability to freezing and thawing were not executed.

Based on interviews with engineers in the area, it was determined that the Weeping Water source (the supplier of the coarse aggregate for Offutt pavement) generally enjoys a reputation for good-quality aggregate. Apparently, there was a problem over 10 years ago with some low-quality material accidentally getting into the product stream, but this was short-lived. Therefore, a judgment of a satisfactory service life for this source would appear to have been appropriate.

After this work was completed and the draft report written and reviewed, another person knowledgeable of aggregate sources in Nebraska was encountered who reported the presence of a seam of unsound material in the roof of the Weeping Water aggregate mine, which occasionally slips

into the product stream. It is suspected that the dolomite aggregate associated with the popouts at Offutt is attributable to this material.

The nondurable dolomite appeared to exist in the Offutt concrete largely as a moderate-sized particle, about 10 mm in diameter. The concrete in Taxiway P seems to be an exception. In that structure, a large number of popouts resulted from aggregate particles 15 to 25 mm in size.

It is absolutely clear that this aggregate is responsible for the observed popouts. However, the direct evidence that it may be causing, or contributing heavily to, the sliver spalling and other damage is less clear.

That such an aggregate is responsible for damage near joints, where concrete saturation typically stays high, is very plausible. The larger particles tend to form popouts when located near a concrete surface and saturated with water. Small pieces of such aggregate could plausibly cause subcritical damage that is later revealed by service loads.

It is also plausible that the larger particles of this aggregate located somewhat deeper in the concrete could cause or contribute to the relatively abundant corner cracking observed on Taxiway P. Visible evidence of this was found in the analysis of Core 19, taken from a damaged corner of a slab from Taxiway P. This damage may be an early form of the aggregate-related damage known as D-cracking.

The earlier ERDC investigation of the problem (Poole et al. 2007) did report the presence of this aggregate type in the fracture zone of a sliver spall provided by the Offutt maintenance team. Some of the monitoring data suggest a nonuniform distribution pattern of this aggregate in the concrete, which may explain the observed pattern of damaged and non-damaged sections of joint in close proximity.

It is also plausible that the use of potassium acetate on this concrete could be contributing to the damage issue beyond the particular aggregate type known to be particularly nondurable.

That nondurable material appeared in the product stream of a source thought to be acceptable suggests there may need to be a review of aggregate acceptance-testing protocols. Many quarries contain variable material. Nonsuitable material is normally excluded by quarry operating

procedures. The normal procedure is to determine the acceptability of a source of aggregate during the design and planning stage of a project, then to assume that all of the properties, except grading, are stable throughout the construction. It is at least an occasional problem in Corps construction that the product stream is not stable, and objectionable material gets into a structure. The consequences of this can vary from a nuisance to a serious problem for the structure.

Air entraining

Project specifications were based on total air as measured in the fresh concrete on delivery to the paving site. Project quality assurance data showed the air content of fresh concrete to be under good control (uniformity) and in compliance with the project specification requirements of $6.0 \pm 1.5\%$. Air contents were generally in the lower half of the allowable range, but no test results exceeded the 4.5% lower limit. Air contents of hardened concrete, as determined by ASTM C 457, indicated an average loss in total air of 1.4%, apparently during placing and consolidation. Total air ranged from 1.4% to 6.7%.

The test methods for air content of fresh concrete and for hardened concrete differ, so the loss of air could be attributable to a bias between these methods. However, there is no general recognition that such a bias exists. On the other hand, loss of air during placing and consolidation is a well-known phenomenon, and it has been known to sometimes result in no loss in effectiveness of the air void system (Backstrom 1958b; Whiting 1983; ACI 1998b).

Even though specifications are usually based on total air, it is widely believed that the structure of the entrained air is more critical than the actual amount. The spacing factor (average distance between air voids) is considered the most important of the metrics of the air void structure.

The amount of literature on spacing factor is rather large. The following references reasonably cover the matter: Powers (1949); Backstrom (1958b); Mielenz et al. (1958); Pigeon and Lachance (1981); Malhotra (1982); Mather (1982); Philleo (1986); Hobbs et al. (1998); Stutzman (1999); and Jeknavorian (2006). The body of work is based primarily on laboratory testing, but it also includes field data and theoretical representation.

There has been some debate over the years regarding the appropriate value of the spacing factor. However, the ACI settled on a limit of ≤ 0.20 mm about 50 years ago, and this has been held as a reasonable value since. Other agencies have similar guidance. Canada uses ≤ 0.23 mm as an average figure, with no single test higher than 0.26 mm. Even though construction specifications are not usually developed around the spacing factor, it is probably prudent to reasonably conform to requirements based on these levels, which are reasonable consensus values taking into consideration all the literature on the matter.

Mean spacing factor for the Offutt concrete was found to be 0.25 mm. The concrete at Eppley Field averaged 0.22 mm, and concrete at Mountain Home AFB averaged 0.24 mm. These latter two structures were not noted for having joint-damage problems.

Of concern in the analysis of the early-age damage of the Offutt concrete is not so much that the mean value (0.25) mm was higher than the ACI guidance of 0.20 mm, but that the variability was so high, and skewed toward high values—indicating that much of the concrete had much higher spacing factors. If one takes the Offutt sampling as random and therefore representative of the entire pavement in Taxiways C and P, then about 80% of the concrete would be concluded to be beyond the ACI limit of 0.20 mm, and about 20% beyond the Canadian single-value acceptance limit (0.26 mm).

Increasing the total air content of the concrete is one way to overcome the questions associated with air loss. Perhaps this would also improve the uniformity of the entrained air properties. The Nebraska Department of Transportation has recently begun requiring higher levels of total air (9 to 10%) in concrete because of the losses that occurred during placing and consolidation. This decision was based on their belief that these losses resulted in some loss of durability to freezing and thawing, and that this damage was promoting a more serious alkali-silica reaction. It is reported that the loss in early strength due to the added air was not a problem in construction.

Another approach to air problems is the Air Void Analyzer (Grove et al. 2006). This instrument is a relatively new technology that allows the user to monitor air void parameters on concrete after placing and consolidation, but before time of setting. This instrument requires a relatively small

sample and gives test results in a few minutes. This technology appears to have the potential to allow the user to investigate the effects of placing and consolidation on the in-place air void structure. This could plausibly be used during construction, or at least during the placement of demonstration structures as part of the overall preconstruction verification operation. The relative ease of execution would allow a reasonably intensive investigation into the uniformity properties of the concrete. This technology has been under investigation for practical application at the National Concrete Pavement Technology Center (Grove et al. 2006), so the development of practice around it should not be a “from-scratch” effort.

Deicing salts

Scaling of concrete in the presence of salts during exposure to cycles of freezing and thawing is an old topic. Marchand et al. (1994) is a good general treatment of the subject.

It was confirmed that potassium acetate deicing salt solutions are used on concrete at Offutt on an occasional basis. The usual practice is to use the salts for ice removal, but sometimes they also sprayed on in anticipation of an icing event. Consequently, the concrete is exposed to the salt, but not continuously so during the winter. Sodium acetate has been used in the past but has now been discontinued due to economic considerations. It is not clear whether the concrete in this study was treated with this salt.

Only one reference (Wang et al. 2006) included potassium acetate specifically, and it was found not to be a significant contributor with the aggregates used in that study. Potassium acetate is a relatively new salt in airfield maintenance practice and may not have been the focus of much analytical work. Still, the fact that the D 5312 tests showed some scaling of the cement paste fraction is not too surprising. It was not determined whether it was worse than expected based on past experience with other deicing salts.

A reasonable hypothesis is that the deicing salts may have had an exaggerated effect because they were applied during the first winter, soon after the pavement was completed. One of the proposed mechanisms for salt scaling damage to paste is that the presence of the salt causes the concrete to be more saturated than when it is absent. It could be argued that the cement fraction of the concrete had not reached a degree of maturity during the first winter sufficient to resist this effect. ACI 201.2R discusses the likely

effects of lack of drying of new concrete on accelerating salt scaling. Potassium acetate salts have been reported to have been in use at Eppley Field under similar conditions, and no damage has been reported from that facility.

The more surprising fact was that the salt seemed to have so much effect on the durability of some coarse aggregate. Dubberke and Marks (1987) reported that calcium chloride deicing salt had a negative effect on frost durability of some dolomite aggregate in Iowa. This may be pertinent here, given the geographical proximity and the fact that some dolomite aggregate types are found in the Offutt concrete. One of these was identified as being particularly susceptible to frost damage.

The deicing study reported here was relatively small, representing a single exposure condition to the salt. However, the exposure condition reasonably represented the field exposure condition in which concrete was exposed for relatively short periods on an intermittent basis.

Monitoring

The results of the monitoring over the second winter after construction showed significant decline in rate of gross damage relative to the first winter. Several things would seem to contribute to this much-reduced rate of damage.

Sections of concrete along joints that were not damaged during the first winter may have been durable to cycles of freezing and thawing, hence not susceptible to damage during the second winter either. This would support the idea of nonuniform concrete with respect to durability to freezing and thawing cycles.

Sections that were susceptible the first winter continued to suffer some damage the second, but not in the same form. Formation of new sliver spalls was rare, but there was some continued eroding of the material from the concrete remaining in previously damaged zones (old spalls were removed prior to the second winter). This may be partially attributable to the change in the cross-section profile of the joint due to the damage during the first winter.

The formation of sliver spalls tended to transform the square cross section of joint into a more rounded, shallow-angle type of surface. This may have

changed the way in which water was held there, also changing the way loading stresses developed on those sections of the joint. Also potentially changing the structure of the joints was the removal of loose damaged material in fall 2007, followed by sealing with a field-cast sealant that tended to fill the joints differently than the original precast seal used in the initial construction.

Perhaps significant is that use of brushes on the snowplow equipment was emphasized during the second winter. It is unlikely that snowplow damage per se was the cause of the damage to the concrete, but it may have played a significant role in exposing the microstructural damage from freezing and thawing of the concrete.

Maturity

Freezing of green concrete and/or overloading of concrete before it gains sufficient strength are, in theory, both plausible mechanisms for the observed damage. The last concrete was placed in mid-October 2006. It is common belief (e.g., Headquarters, U.S. Army Corps of Engineers 2001) that concrete must have a compressive strength of at least 500 psi before a single freezing event occurs and at least 3500 psi to withstand cycles of freezing and thawing. Paving concrete normally develops more than 500 psi during the first 24 h after placing and reaches 3500 psi after a few days, perhaps a week, at moderate temperatures. However, the temperatures at Offutt AFB soon after the last placement were relatively cold, with significant freezing events and low average temperatures.

The maturity analysis done in this work gives an approximation of the likely effects of the cold weather that developed after the last placements of concrete. It was assumed that the average concrete temperatures in the concrete are reasonably represented by the average air temperatures. This might not have been the case in the first day or so, given the thickness of the placements (about 600 mm), although it was probably reasonably true later.

The last two dates of placement (17 and 19 Oct 2006) were included in the analysis. A freezing event of 23 °F did occur on 19 Oct, presumably in the early morning before the placement on that date. The concrete was estimated to have been between 24 and 48 h old at that time. The maturity calculation projected strength to be between about 400 psi (24 h) and 1200 psi (48 h). It seems reasonable that the strength probably was over

500 psi at that time, particularly given the contribution of the early-age heat of hydration that is not accounted for in the calculation. The guide specification (UFGS 2006) requires protection during the first 72 h, apparently as a precaution.

The estimated strength when opened to traffic was about 4,000 psi. This estimate is probably a little low, again because of the early heat of hydration boost to strength that was not covered by the calculation. We believe this strength would have been sufficient to the first service loads.

A significant point is that almost all of the concrete along the centerlines was placed well before the onset of the cold weather experienced by the last placements. This further supports the hypothesis that lack of maturity was not a factor in the observed damage.

Integration of individual damage concepts

Three factors were identified in this study as being likely contributors to the damaged observed at Offutt after the first winter: (1) much of the concrete having poorer entrained-air systems (in spite of meeting project specifications on total air; (2) presence of nondurable coarse aggregate; and (3) action of the potassium acetate deicing salt on the paste and aggregate in the concrete.

It was not possible at this point to estimate the relative contributions of these three factors, although it is plausible that they do vary in importance. That the damage was not uniform is perhaps a significant indicator of the probable way these factors interacted. Even on the badly damaged centerline joint of Taxiways P and C, there were sections of the joint that were not damaged at all during the first or second winter.

Plausibly, major damage occurred when two or more of these factors coincided. There was evidence that both unsound aggregate and poorly air-entrained concrete were quite variant properties over the structure. It is plausible that where both of these coincided, together with the more uniform exposure to deicing salt, a synergistic effect developed, resulting in more damage than would have been expected from each of them alone.

Given the damage mechanisms identified in this work, it would be expected that damage would have appeared about equally on all joints—given the propensity of joints to hold water and keep nearby concrete

saturated. That the major damage occurred on the centerlines, which were also the high spots in the pavement, suggests that service loads played a role. Wide experience has shown that kinds of service loads normally applied to this pavement are of a type that does not cause significant damage to sound concrete. However, it is plausible that concrete damaged by the above mechanisms could be revealed by service loads.

5 Conclusions and Recommendations

Conclusions

The field samples of concrete taken from the Offutt AFB runway and Taxiways P and C were all determined to be not durable to repeated cycles of freezing and thawing as tested according to ASTM C 666. The specimens showed significant damage both to cement paste and coarse aggregate particles.

Some significant issues were identified in the air entraining. Loss of total air content of 1.4% between delivery of fresh concrete and completion of placing and consolidation was documented. Analysis of air void parameters in hardened concrete showed that the average value of the important parameter, spacing factor, was larger on average than recommended by ACI guidance. More importantly, the variation was high enough that it was concluded that a significant fraction of the concrete may have insufficient air void development. It was concluded that the air void deficiencies could plausibly have led to microcracking, or to more significant damage of the concrete during the first winter.

Presence of significant amounts of frost-susceptible coarse aggregate was identified. One particular aggregate type that was identified—a very light-colored dolomite—is clearly showing evidence in field inspections of causing damage. The aggregate is definitely causing numerous popouts in the surface of the concrete and is plausibly contributing to the joint spalling problem. Also, some evidence was found that it may also contribute to the corner fracturing of slabs.

Both cement paste and some coarse aggregates were found to be nondurable to cycles of freezing and thawing after relatively brief exposure to 50% potassium acetate solution.

Damage in the second winter was considerably less than in the first. Significant damage that was identified as having occurred during the second winter was located primarily in the zones along the centerline joint that were already damaged during the first winter.

Some combination of air-entraining deficiencies, nondurable aggregate, and deicing salt exposure was concluded to be ultimate causative factor for the damage. Mechanical service loads and/or environmental conditions were plausibly involved in revealing the damaged concrete by causing spalls of previously damaged concrete to form.

A small amount of concrete placed in mid-October probably experienced a significant early freezing event, although it was determined that this probably was not critical. Strength development at the time of opening to traffic was believed to be significantly retarded due to cold weather. However, it was concluded that adequate strength had developed by the time of opening to traffic. Given that most of the concrete involved in the damage under study was placed well before these late-season placements, it was concluded that lack of adequate maturity was not involved in causing the sliver-spall damage.

Recommendations

Protocols should be developed for investigating the air void parameters of the in-place concrete to verify that the total air content of the fresh concrete is an adequate predictor of in-place air, and that air void structure is not affected by placement and consolidation. Particular attention should be given to variability of in-place air. Highly variable in-place air might require higher air contents in fresh concrete to compensate for the lower values found in the in-place air.

This might be cumbersome with existing test methods, but possibly tractable with some modifications to practice. A relatively new technology, the air void analyzer, has been shown to offer promise for this type of investigation because of its field applicability and short testing-time requirements.

Another approach to the air problem would be to increase the level of required air in the concrete to compensate for any losses during placing and consolidation. The Nebraska DoT has worked with this concept and reports success in developing concrete with improved durability. We do not believe that excessive loss of air due to placing and consolidation has occurred in USAF pavements, because of the relatively low number of reports of damage attributable to this phenomenon. Therefore, requiring a USAF-wide change might be too strong a response.

The current guidance on aggregate acceptance testing should be reviewed for ways to avoid unintended contamination of an aggregate product stream. The current protocol allows for a service-record exemption from laboratory testing. This practice seems to have been acceptable in most cases, but some type of quality assurance procedure needs to be developed to detect exceptions.

Past experience with such exceptions has been that the contaminating material is something that is a known feature of the quarry, but normally avoided by selective quarrying. Appropriate procedures to identify a potential problem would probably vary among sources, depending on the details of the quarry. This would probably require detailed evaluation of quarries by a qualified geologist.

A more detailed investigation is recommended to determine whether the newer deicing salts create a worse durability situation than has been documented for the traditionally used salts. A test method based on ASTM D 5312 may be useful for this.

Finally, we recommend that consideration be given to use of the maturity method for tracking strength development of in-place concrete when the temperature is below 10 °C for significant periods, if this is not part of current practice. Hardware and software are commercially available for executing this type of analysis. Also, commercial vendors of engineering services can perform this kind of work.

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14. ABSTRACT This report summarizes an investigation into the cause of excessive sliver-spall damage in new airfield concrete after first winter exposure in Nebraska. The damage was particularly concentrated along joints that experienced more service loads than average; however, other sections of high-load joints showed no damage. The results showed that a combination of deficiencies in air entraining, sporadic occurrence of nondurable coarse aggregate and, possibly, early application of deicing salts contributed to the damage. The amount of new damage was notably lower after the second winter of exposure. Several recommended revisions to current concrete practice were made.					
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