
**Airfield Asphalt Pavement
Technology Program Project 05-03:
Effect of Deicing Chemicals on HMA
Airfield Pavements**

REVISED FINAL REPORT

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ABSTRACT

The purpose of AAPT Project 5-3 was to investigate the performance of HMA airfield pavements subjected to deicing and anti-icing chemicals (DIAIC). The most commonly used DIAICs include potassium acetate, sodium acetate, urea, and ethylene and propylene glycol. Recently, several Nordic countries have reported what appears to be damage in HMA pavements related to the use of DIAICs. The exact mechanism of this damage is not clear, although it appears to be a form of moisture damage accelerated by the low surface tension and relatively high density of many DIAIC solutions. DIAIC-related damage does not appear to be common in airfield pavements in the U.S. and Canada. If it is suspected that an HMA mixture is susceptible to DIAIC-related damage, a simple procedure called the immersion tension test can be used to perform an evaluation. In cases where DIAIC-related damage is a problem, mixture performance can be improved by using a stiffer binder and/or by incorporating hydrated lime into the mixture. When an HMA mixture prone to DIAIC-related damage is used in an airfield pavement, it is essential to thoroughly compact the pavement to reduce the air voids to as low a level as practical.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

OBJECTIVE

The Project 05-03 Request for Proposal provided the following statement of the project objective:

The objective of this study is to investigate the performance of HMA airport pavements subjected to deicing and anti-icing chemicals that are being used to minimize the effect of snow and ice on aircraft and airport pavement facilities. This study shall have both a field component and laboratory component. The field component will gather and evaluate information on the usage of deicing and anti-icing materials and investigate damaged and undamaged HMA airfield pavements that have been subjected to these chemicals. The laboratory component will propose laboratory test methods and procedures to identify the potential for damage to HMA in the mix design process in order to prevent or minimize the impact on HMA airport pavements. The product from this effort shall be a comprehensive document of field performance and guidance to the Federal Aviation Administration (FAA), draft specifications, FAA engineering brief (EB) or technical advisory circular (AC)...

RESEARCH APPROACH

The research included several components, as described in the objective for Project 05-03. The work was organized into two phases and 11 separate tasks:

Phase I

Task 1: Review Existing Literature

Task 2: Collect Information on Usage of Deicing and Anti-Icing Chemicals

Task 3: Collect Initial Field Performance Information

Task 4: Conduct Preliminary Laboratory Tests

Task 5: Develop Detailed Phase II Plans

Task 6: Meet With Project Panel

Phase II

Task 7: Conduct Field Investigation

Task 8: Conduct Laboratory Testing

Task 9: Compile Interim Findings

Task 10: Submit Draft Final Report

Task 11: Submit Revised Final Report

Task 1 entailed a review of existing literature. Publications on deicing and anti-icing chemical (DIAIC) use on airfield pavements are very limited. The available literature was reviewed during the course of the project. Tasks 2 and 3 were closely related, involving collection of information on the use of deicing and anti-icing chemicals at airports in the U.S. and Canada, and collection of information on the performance of HMA pavements at these facilities.

Three laboratory tests were identified for potential use in identifying DIAIC-related damage in HMA pavements: (1) the ultrasonic horn test, as developed at the Western Research Institute (WRI); (2) a variation of the modified Lottman Procedure (AASHTO T-283); and (3) a long-

term durability test specifically developed for use in this project. In Task 4 of this project preliminary testing was performed in order to evaluate these three procedures. The test based upon AASHTO T-283 appeared to be most effective in evaluating DIAIC-related damage in HMA mixes, and was the basis for many of the laboratory tests to be performed during Phase II of the project. As discussed in the body of this report, this procedure was refined during AAPT Project 5-3, and is referred to as the immersion tension (IT) test.

Tasks 5 and 6 consisted of developing interim findings based upon the Phase I work, developing a plan for Phase II of the project based upon these findings, and presenting these findings and plans to the project panel. This presentation was given at the end of February 2007; following review by the panel and some modifications to the Phase II plans, work commenced on Phase II of AAPT Project 5-3.

In Task 7, site visits were made to four airfields: Boston Logan International Airport, Boise Idaho Airport, Hailey Idaho Airport and Colorado Springs Municipal Airport. These airports were selected because preliminary investigations indicated that they were likely candidates for DIAIC-related damage. Cores were taken from various locations in pavements at these facilities for testing as part of Task 8. The bulk of the activity in Task 8 involved evaluating a variety of HMA mixtures to determine if they were susceptible to DIAIC-related damage, and also to determine the effect of various factors on any observed DIAIC-related damage. Factors studied included air voids, use of hydrated lime, and binder grade.

Submission of this Draft Final Report constitutes Task 9 of AAPT Project 5-3. After AAPT review, Task 10 will involve preparing the Revised Final Report.

The research team for AAPT Project 5-3 consisted of personnel from three organizations: Advanced Asphalt Technologies, LLC (AAT); Applied Research Associates (ARA); and the Western Research Institute (WRI). AAT was primarily responsible for Tasks 4, 5, 6, 8, 9, 10 and 11. ARA was primarily responsible for Tasks 1, 2, 3 and 7. WRI performed laboratory testing and analysis under Tasks 4 and 8. Dr. Donald Christensen of AAT was principle investigator for the project; Mr. David Hein had primary responsibility for activities performed by ARA, and Mr. Mike Farrar had primary responsibility for work performed by WRI during Project 5-3. Important contributions to the research effort were made by Dr. Ray Bonaquist of AAT, Mr. Jagannath Mallela of ARA, and Dr. Eric Kalberer of WRI.

ORGANIZATION OF THIS REPORT

Chapter 2 of this report presents the findings and primary analysis for the various activities that took place during this research. Information presented includes the results of the literature review, the Phase I preliminary laboratory testing, field surveys of airports, and the Phase II laboratory testing. Detailed information on some of the more complex laboratory procedures, such as surface tension measurements, are presented in appendices, as are detailed reports of airfield site visits.

Chapter 3 is a brief discussion of the overall significance of the findings presented in Chapter 2. Chapter 4 presents the final conclusions of AAPT Project 5-3, while Chapter 5 is a list of references for the report.

The report includes four appendices. Appendix A contains detailed reports, including numerous photographs, for the site visits made by ARA during the project. Appendix B is a detailed report on Fourier Transform Infrared Spectroscopy (FTIR) analyses performed by WRI. Appendix C is a detailed report on surface tension measurements, also performed by WRI during the project. Appendix D is a draft specification in ASTM format for the immersion tension (IT) test, developed during AAPT Project 5-3 as a tool for evaluating susceptibility to DIAIC-related damage in HMA mixtures.

CHAPTER 2

FINDINGS AND ANALYSIS

In this chapter descriptions are presented of the various activities performed during APTP Project 5-3. The chapter follows the project organization listed in Chapter 1. The chapter begins with a description of information concerning DIAIC-related damage in airfield HMA pavements found during the literature review. This is followed by a summary of the results of a survey of airfield facilities on DIAIC usage. The Phase I preliminary laboratory testing is then discussed, followed by a description of site visits made to several airports to investigate potential DIAIC-related damage and to take pavement cores for testing during Phase II of APTP Project 5-3. Phase II test results, including testing of these field cores, are presented last in this chapter.

SUMMARY OF LITERATURE ON EFFECT OF DEICING/ANTI-ICING CHEMICAL ON HMA AIRFIELD PAVEMENTS

During APTP Project 5-3, a comprehensive domestic and international literature search was conducted to identify information related to the utilization of deicing/anti-icing chemicals used on aircraft and on airfields and their potential adverse impact on the performance of airfield asphalt pavements. The ultimate goal was to recruit airports in the United States (U.S.) and Canada with asphalt pavement features (i.e., runways, taxiways, aprons, etc.) that have been subjected to deicing and anti-icing chemicals and which are exhibiting distresses suspected to be caused by these chemicals.

The sources of information sought in the literature review process included the following:

- Review of previous research conducted by the Innovative Pavement Research Foundation (IPRF) on a similar topic but for concrete airfield pavements, and dedicated deicing facilities.
- Transportation Research Information Systems (TRIS).
- American Society of Civil Engineering (ASCE) publications databases.
- General internet searches.

In addition, U.S. Federal Aviation Administration (FAA) regional offices, Transport Canada, Public Works and Government Services Canada, the U.S. and Canadian Air Force officials, deicing chemical manufacturers, and airport managers and airport operation/ maintenance superintendents at individual airports were contacted to determine their experience related to the use of deicing chemicals for asphalt pavements. A listing of airfields where funding has been provided by the FAA to construct dedicated deicing facilities in the past 10 years was also obtained from FAA's Engineering Division office.

The literature search revealed numerous research studies, papers, publications, and specifications on practices related to the use of deicing/anti-icing chemicals on aircraft, airfield and highway pavements. However, as expected, a majority of these studies were related to the environmental impact and not related to the impact of chemicals on airfield pavements. Some of the closely related literature on the impact of deicing chemicals on pavements includes the work of (*USEPA 2000; Cryotech.com; Nilsson 2003; Olek and Rangaraju 2006; Air Force 2002; Farha et al.*

2002; Hassan. et al. 2001; Van Dam et al. 2003; Alatyppo 2005). The information from these studies is summarized in the following paragraphs.

Deicing/Anti-Icing Practices

Based on a review of literature, it was found that, for most part, chemicals used for deicing and anti-icing practices are the same. Generally speaking, deicing is defined as the process of removing ice and snow from the airfield pavements or aircraft, therefore deicing is considered a reactionary operation. In contrast, anti-icing is referred to as a proactive operation i.e., surface treatment of airfield pavements or aircraft prior to ice or snow formation. Anti-icing eliminates ice accumulation or facilitates ice or snow removal by reducing the bond between the surface and the ice (*USEPA 2000*). As noted above, the acronym DIAIC is generally used throughout this report to mean deicing and/or anti-icing chemicals for pavement usage.

According to the U.S. Environmental Protection Agency (*USEPA 2000*) and a review of similar literature on deicing/anti-icing practices, the only difference between anti-icing and deicing fluid used for aircrafts is the small amount of a proprietary thickener that is added during anti-icing for increased holdover time. For simplicity sake and ease of reading, the term deicing has been used throughout the report to represent both deicing and anti-icing operations.

Aircraft Deicing Practices

Aircraft deicers are categorized into four general classes: Type I, Type II, Type III, and Type IV. Not all types are currently used (Type II, and type III are being discontinued). Fluid types vary by composition and allowed holdover times (i.e., the amount of time the residual fluid will protect an aircraft from ice formation). Type I is the most commonly used fluid and is used primarily for aircraft deicing (*USEPA 2000*). Type I fluids are commonly purchased as concentrated propylene or ethylene glycol solutions (8% water, 90% glycol, and less than 2% additives) and diluted with additional water depending on the ambient temperatures.

Type I fluids have between 6- and 15-minute holdover time in a light snow. Because of this brief time span, Type I fluids are used for deicing, and for only short-term anti-icing protection. A typical Type I deicing fluid contain approximately 50% to 60% glycol after being diluted for application (*USEPA 2000*). For longer holdover times, either Type II or Type IV anti-icing solutions are applied, effectively extending holdover times to about 30 minutes and 80 minutes, respectively. Anti-icing fluids Type II and IV are sold pre-formulated, to the appropriate concentration (33% water, 65% glycol, and 2% additives) and do not require dilution prior to application (*USEPA 2000*). Type II and Type IV solutions contain a polymer thickener to enhance adherence to the aircraft surfaces (*USEPA 2000*).

Airfield Pavement Deicing Practices

Based on the literature reviewed and the information obtained from interviews with airport operation managers and airport superintendents at thirty-six (36) airfields across the U.S. and Canada, the most predominant deicing chemical used presently for airfield pavements is potassium acetate. In the past, urea and glycol based materials were used as the chemicals of choice for deicing, and anti-icing of airfield pavements, and a limited number of airports are still using such chemicals. In recent years; however, there has been a dramatic shift towards the use of acetate-based deicers, in particular potassium acetate. Overall, more than 90 percent of the

airports interviewed use potassium acetate; a majority of these airports were from the northern tier of the North American continent. Other acetate and formate based chemicals used by some airports include sodium acetate, potassium formate, and sodium formate. However, a limited number of airports are still using urea and only one airport interviewed reported using ethylene glycol as a pavement deicing chemical.

The rate of application for potassium acetate in anti-icing operations is typically 0.5 gallon per 1000 ft² and in deicing operations, depending on the amount of ice and snow, ranges anywhere from 1 to 3 gallon per 1000 ft² (*Cryotech.com*).

The primary reason for the increased use of acetate based deicers in recent years appears to be because they are environmentally friendly, requiring much lower biological oxygen demand (BOD) to decompose, and are much less toxic than the traditionally used urea and glycol based chemicals. Based on interviews with airport officials in the U.S. and a review of the experience with Nordic Airfields (*Nilsson 2003*) it appears that the use of acetate-based deicers is not without concerns. There is concern that these deicers damage airfield pavements and associated infrastructure such as airfield lighting systems. The purpose of this study is to evaluate the potential impact of deicing chemicals on the performance of asphalt pavements.

The impact of deicing and anti-icing chemicals on concrete pavements have been studied for many years, and it appears that their use have caused or accelerated materials related distresses (MRD) such as freeze-thaw deterioration of hardened cement paste and alkali-silica reactivity (ASR) (*Olek and Rangaraju 2006*). MRD-like distresses observed at two major airports in Colorado, Denver International Airport (DIA), and Colorado Springs Airport (COS) have prompted the Division of Aeronautics of the Colorado Department of Transportation (CDOT) to initiate a major study looking into the causes of these distresses.

Impact of Deicing/Anti-icing Chemicals on Airfield Asphalt Pavements

Application of DIAICs to runways and taxiways is required by the FAA to ensure the safety of passengers and aircraft. These chemicals play a major role in the overall operation of cold region airports during the winter months. Unfortunately, chemical agents used in deicing and anti-icing processes impact natural resources by depleting the available oxygen in the receiving waters *and* their discharge into the environment is subject to stringent control by the U.S. EPA (*Air Force 2002*). In addition to environmental impact, concerns have been emerging on the possible airfield asphalt pavement damage caused or accelerated by these chemicals. The few studies that relate possible adverse impacts of deicing chemicals to performance of airfield asphalt pavements are summarized in the ensuing paragraphs.

Recent Studies of Deicer Damage to HMA Pavements

In the 1990s, asphalt durability problems potentially caused by the use of new deicing chemicals were observed at some Nordic airports. Degradation and disintegration of asphalt pavements occurred and there was also softening and stripping effects on bitumen and asphalt concrete together with loose stones on runways. These problems occurred when airports in Norway and Sweden changed from urea to potassium acetate and potassium formate (*Edwards et al. 1999, Nilsson 2003*).

Researchers at The Swedish National Road and Transport Institute (VTI) completed a study on DIAIC-related damage at Nordic airfields in 1999 (*Edwards et al.*). This research was a joint effort funded by a number of agencies representing various Nordic countries, and primarily performed by the VTI. The Nordic research team observed that acetate- and formate-based DIAICs tend to soften asphalt binders and degrade asphalt concrete mixtures. The effect appeared to increase with elevated temperatures, increasing pH of the DIAIC solution, and decreasing binder stiffness. These researchers identified several potential mechanisms for the DIAIC-induced damage: (1) neutralization of carboxylic acids within the asphalt binder, resulting in a weaker asphalt-aggregate bond; (2) weakening of the asphalt-aggregate bond due to the high pH of DIAIC solutions; and (3) dissolving of acidic minerals (such as silica) at the aggregate surface by the DIAIC solutions, resulting in a weaker asphalt-aggregate bond. The Nordic research team pointed out that because formates and acetates are highly hygroscopic (attracting water), pavements containing formate- and acetate-based DIAICs will tend to retain moisture longer than pavements not subjected to deicing chemicals. Furthermore, DIAICs also tend to lower the surface tension of water, making it easier for moisture to penetrate into an asphalt concrete mixture where it can cause damage. The VTI publication listed DIAICs in the order of increasing tendency to damage HMA pavements as follows: urea, calcium-magnesium acetate (CMA), other acetates, and finally formates.

The Nordic research has resulted in two tests for evaluating potential DIAIC-related damage in HMA pavements (*Edwards, 1999*). The first test is an asphalt binder test, in which the softening point is determined before and after storage in DIAIC solution. The second test is a mixture tension test. In this procedure, a small cylindrical area is pulled away from the surface of an asphalt concrete specimen (a field core or slab, or a specimen compacted in the laboratory). The surface tensile strength is determined in this way on unconditioned specimens and specimens conditioned with one or more DIAIC solutions. The procedure is somewhat complicated and requires specialized equipment, including a servo-hydraulic test system. However, it is perhaps convenient to run in the Nordic countries since it is a modification of a standard procedure in Sweden (and perhaps other Nordic countries) for measuring the adhesion of pavement marking materials to HMA pavement. In the original research, one set of specimens was tested without any conditioning and another set after vacuum saturation and storage at 40°C for 14 days in DIAIC solution. This procedure did result in some decrease in measured surface tensile strength for specimens conditioned in DIAIC solutions compared to those tested with no conditioning. In follow-up research on the repeatability of this test procedure, the conditioning time was increased to 70 days. A variety of DIAIC solutions were evaluated, along with de-ionized water and no conditioning (air). Interestingly, only specimens conditioned in sodium formate showed lower surface tensile strengths than those conditioned in water. In all the Nordic research, relatively high concentrations of DIAICs were used—in the repeatability study, concentrations ranged from 32 to 52.9 % by weight (*Ekblad and Edwards, 2001*). The initial Nordic study evaluated the indirect tension test (IDT) only in an experiment involving pavement slabs subjected to DIAIC application in a weather simulator. In this case, the IDT test apparently did not show any decrease in strength after conditioning. However, neither did the surface tension test. The IDT test was not used in conjunction with the long-term, high temperature conditioning in DIAIC solutions that was used successfully with the surface tension test. Therefore, it is not clear based on the Nordic research that the surface tension test is in fact superior to the IDT test

for evaluating DIAIC-induced damage. Also, virtually no emphasis was placed on aggregate type or DIAIC concentration in the Nordic research.

In 2002, Farha and his co-authors presented a study on the effects of various DIAICs on asphalt concrete pavements at Canadian airfields (*Hassan et al. 2001; Farha et al., 2002*). It should be pointed out that this study was not initiated because it was believed that DIAIC-induced damage was suspected to be a problem in Canadian airfields, but rather because airports in Canada had recently changed from urea to newer compounds such as sodium formate, potassium acetate and sodium acetate, and there was concern that these new DIAICs might cause damage in HMA pavements. In the initial phase of this research (*Hassan et al. 2001; Farha et al., 2002*), aggregate samples and HMA cores were subjected to freeze-thaw cycling and evaluated for signs of damage. This evaluation was based on weight loss or gain, changes in indirect tensile strength and elastic modulus, and change in penetration of asphalt binder recovered from the conditioned cores. For the most part, there was little or no evidence of DIAIC-induced damage in any of the test specimens. However, cores subjected to freeze-thaw cycling in urea solutions began to disintegrate after 50 cycles.

A follow-up study (*Farha et al. 2002*) used similar techniques as the original study, but included wet/dry cycling at 40 °C in addition to freeze-thaw cycling. The researchers felt that elevated temperatures might accelerate potential chemical reactions occurring between the DIAIC solutions and the asphalt binder. As in the first study, there was little evidence of significant DIAIC-induced damage.

Although, the Canadian studies suggest that newer compounds used as DIAICs on airfield pavements caused little or no damage to HMA mixtures, care should be used in interpreting these results. Only one HMA mix was used; it is well known that moisture damage in HMA is strongly related both to binder source and aggregate type. It is possible that the results of this study would have been different if a broader range of binders and aggregates were used. It is also possible that the high temperature cycling used in these studies was not high enough to produce the reactions needed to cause DIAIC-induced damage.

Many studies have been conducted on the effects of newer DIAICs on concrete pavements. Recent investigations (*Olek et al. 2006*) have indicated increased occurrence of premature deterioration in concrete pavements, particularly due to alkali-silica reaction (ASR). Based on this research, it was found that the two common airfield DIAICs, potassium acetate and sodium acetate, have a significant potential to cause ASR in test specimens containing reactive aggregates. ASR reactions in HMA containing reactive (siliceous) aggregates exposed to certain types of DIAICs that are popular today cannot be ruled out as a potential contributing factor in moisture-induced damage.

The impact of glycol-based chemicals on concrete at the dedicated deicing facilities (DDFs) was studied by Van Dam and his associates (*Van Dam et al. 2003*). The study concluded that no common cause of distress could be assigned to the concrete at the DDFs. Furthermore, no evidence was found to suggest that the uses of glycol-based aircraft deicers are directly implicated in the chemical or microbial degradation of concrete.

A recent study on the effect of DIAICs on HMA pavements was conducted by the Laboratory of Highway Engineering at the Helsinki University of Technology in Finland (*Alatyyppo 2005*). The project was initiated due to the perception that the newer DIAICs such as acetate and formates were causing damage to HMA pavements in Finland's airfields (*Alatyyppo, V. 2005*). The researchers performed experiments on asphalt binders, aggregate and asphalt-aggregate systems. They also completed a variety of chemical and physical tests on cores extracted from in-service airport pavements. Based upon a difficult to read English summary of this research, the key findings included the following:

- After boiling asphalt binders in DIAICs, chemical analyses showed no evidence of chemical reactions between the asphalt binder and deicing compounds.
- DIAICs appear to reduce the surface tension of water, which in some cases might accelerate moisture damage in HMA pavements.
- Aggregate tests showed little evidence of direct damage caused by DIAIC solutions, though there was some evidence of "neutralization" of acidic aggregates by DIAIC solutions, suggesting that DIAICs might accelerate moisture damage in HMA made with acidic aggregates, which would include siliceous aggregates such as sandstone and quartzite, and many igneous rocks.
- Boiling tests of asphalt-aggregate mixtures did result in increased damage of the asphalt binder coating on the aggregate, along with generation of polycyclic aromatic hydrocarbons (PAHs). This suggests that elevated temperatures of 70 to 100 °C might be necessary in order for DIAIC solutions to cause or contribute to damage in asphalt pavements.
- Static and dynamic indirect tensile tests showed no significant difference between specimens conditioned in distilled water versus DIAIC solutions, both in simple soaking experiments and when conditioned using freeze-thaw cycling.
- Evaluation of actual airfield pavements seemed to suggest that formate compounds were, in general, more damaging than acetate compounds. Pavements subjected to formate and exhibiting damage had a foul odor, and when tested using gas chromatography revealed the presence of PAH. These field evaluations also suggested that stiffer binders, especially polymer-modified binders, were more resistant to DIAIC-induced damage than softer, unmodified asphalt binders.
- The Finnish researchers were particularly concerned with resurfacing of pavements previously subjected to DIAIC use. In such cases, the presence of DIAICs in the pavement, along with the very high temperatures associated with paving construction, appeared to lead to accelerated pavement damage.
- Some concern was voiced by the Finnish researchers that PAH are toxic (carcinogenic) compounds that could represent an environmental and/or health hazard. Although these compounds are not normally soluble in water, they can be rendered soluble in acetic acid solutions, and perhaps in formic acid solutions.

These research projects considered together appear to paint a somewhat confused and potentially contradictory picture of the potential for DIAIC-induced damage in HMA pavements.

Laboratory testing has often not indicated that DIAIC solutions significantly accelerate moisture damage. Some research—particularly that conducted in Finland and especially in Sweden by VTI—suggests that conditioning in DIAIC solutions at higher temperatures can in some cases lead to damage to asphalt binders and mixtures. However, all studies to date have been somewhat limited in scope. The important issue of how aggregate type affects DIAIC-related damage has hardly been addressed by any researchers. Many questions remain concerning the potential for deicing chemicals to damage HMA pavements.

Although the Finnish study and the Nordic study did observe significant moisture damage in pavements subject to DIAIC solutions, it is not certain that this is a cause-and-effect relationship and that damage may have occurred even without the use of DIAICs. Results of the IT test performed during AAPT Project 5-3, for example, showed that the tensile strength of specimens conditioned in DIAIC solutions were in many cases similar to or even higher than companion specimens conditioned in water. For the apparent DIAIC-induced damage in HMA pavements, the DIAIC usage may merely be coincidental to a pavement prone to moisture-induced damage. Alternately, it may be that deicer usage on HMA pavements might, in some limited cases, contribute somewhat to moisture induced damage, but is not the primary cause of such distress. In this case the term “DIAIC-related damage” would be more appropriate than “DIAIC-induced damage.” It appears likely that some of the inconsistencies in the results of these research projects are the result of not adequately addressing the effect of aggregate type on DIAIC-related damage in HMA pavements.

Because the summary of findings published on the Finnish DIAIC study was brief and therefore omitted many technical details, and was also difficult to understand, Dr. Terhi Pellinen was included as a consultant on this project to review the Finnish research and provide a more thorough and useful summary. In the interest of brevity, the complete summary is not included in this report. Her own summary, with some minor editing, is presented in the two paragraphs below. It should be noted that Dr. Pellinen’s review was based not only on the original research report as published in Finnish, but on personal interviews with key personnel involved in both the original study, and follow-up research still being conducted on this problem.

One of the aims of the Finnish study was to investigate the damage mechanisms of pavements exposed to deicing chemicals. Based on this research, the high density and low surface tension of deicing chemicals facilitates the penetration of these chemicals into the bitumen and the bitumen-aggregate interface. In addition, deicers are highly hygroscopic causing the exposed pavement to stay wet which in turn makes pavements more prone to moisture damage. The analysis of hydrocarbon compounds in pavements subjected to deicing chemicals suggests that some chemical changes take place in the bitumen exposed to chemicals. Increased PAH compound contents were detected in many different experiments. The increase in PAH compounds was associated with the heat peak of the exposed pavement. The PAH-compounds are carcinogenic and very poisonous but they will not dissolve to water.

Another important finding of the Finnish research is that the application of significant heat appears to be required in order for deicing chemicals to damage HMA pavements. Field experiments at two test sites showed different rates of damaging for acetate and formate. In one of these test areas damage was visible after four years of construction. However, if acetate- and formate-based deicing chemicals are used together, it is

possible that the PAH-compounds are disassociating, forming acetic acid which may allow PAH-compounds to migrate to the environment and ground water. It is not known if the disassociation of PAH-compounds also creates formic acid. Laboratory tests of contaminated pavements suggested that the deicing chemicals were migrating from the contaminated layer to the new pavement layer above. However, this may have been a result of contamination during specimen preparation.

The main thrust of research conducted in Finland since the original study on DIAIC-related damage in HMA pavements appears to be two-fold. First, it has become clearer that this problem is real, and is in essence an acceleration of moisture damage in HMA pavements. Second, the Finns have proposed that this acceleration is primarily the result of (1) a reduction in the surface tension of water caused by deicing chemicals; and (2) the hygroscopic nature of deicing chemicals.

Possible Mechanism for DIAIC-Related Damage

Based on the review of existing literature, it appears that there are several possible mechanisms for DIAIC-related damage in HMA Pavements:

1. DIAICs may decrease resistance to moisture damage in some asphalt/aggregate systems by decreasing the surface tension of the water and promoting attack of the asphalt – aggregate interface.
2. DIAICs may accelerate moisture damage in some systems because their hygroscopic nature causes HMA pavements to retain moisture for longer periods of time.
3. DIAICs may attack the asphalt-aggregate bond, either by neutralizing carboxylic acids within the asphalt binder, or by attacking acidic minerals (such as silica) at the aggregate surface.
4. DIAICs may decrease the resistance of aggregates to damage caused by alternate cycles of wetting/drying and freezing/thawing.
5. DIAICs may cause a softening of asphalt binders, decreasing HMA stiffness and strength and increasing the severity of other forms of distress, including moisture damage. This affect appears to be most pronounced with softer binders.
6. DIAICs may accelerate age hardening in HMA pavements.

It is possible that any number of these mechanisms may work simultaneously in some situations. Overall, based on the literature review and damage mechanism discussed above, a number of factors might affect DIAIC-related damage of HMA pavements:

Aggregate type—HMA made using calcareous aggregates, that is, limestone, dolomitic limestone, and dolomite tends to be resistant to moisture damage. HMA containing siliceous aggregates such as sandstone and quartzite, or igneous rocks such as granite and diabase tend to be prone to moisture damage. Because DIAIC-related damage appears to be a form of moisture damage, it should be expected that a similar trend will be observed in this type of distress.

Asphalt binder chemistry—it is known that moisture damage in HMA is dependent on asphalt chemistry, but it is difficult to predict what specific binders will be prone to moisture damage. Research conducted in Finland and Sweden suggests that in general stiff binders may be more resistant to DIAIC-related damage compared to softer binder grades. Research in Sweden also

appears to show that deicer-induced damage is related to asphalt binder chemistry, but the nature of this relationship is not clear (*Edwards et al., 1999*).

Asphalt binder modification—HMA containing polymer-modified binders may be more resistant to DIAIC-related damage.

HMA permeability—it is likely that less permeable HMA mixture to be less susceptible to DIAIC-related damage, since the deicer solution will not permeate the mix as rapidly as in more permeable mixes. One important measure of permeability is in-place air voids; as air voids increase, permeability increases significantly. HMA mixes with in-place air voids below about 6% generally have low permeability. Permeability also decreases with decreasing maximum aggregate size and increasing mineral filler content.

Deicer type—the findings of the Finnish study (*Alatyyppo 2005*) suggests that sodium formate is more damaging to HMA pavements compared to acetate deicers, such as sodium acetate and potassium acetate.

Deicer Concentration—Canadian research indicates that the most damaging concentration of DIAIC solutions is about 2% by weight.

Temperature—the Finnish study (*Alatyyppo 2005*) also suggests that high temperatures (70 to 100°C) are needed in order for DIAIC-related damage to occur in HMA pavements.

USE OF DEICING AND ANTI-ICING CHEMICALS ON AIRFIELD PAVEMENTS IN THE U.S. AND CANADA

To identify specific airport projects in the United States and Canada for detailed investigation, a review of civilian (commercial and general aviation [GA]) and military airports in the U.S. and Canada that (1) use DIAICs extensively and (2) contain HMA airfield pavements on their runways, taxiways, or aprons was completed. The review included information obtained from the following agencies/sources:

- FAA regional offices.
- U.S. Army Corps of Engineers (Center of Expertise and District offices).
- Canadian Civilian Airport Authorities including Transport Canada (Ottawa and Prairie and Northern Regions) and Public Works and Government Services Canada (PWGSC)
- Canadian Airport Pavements Technical Group (CAPTG) through the Summer Winter Integrated Field Technologies Association (SWIFT)
- Canadian Department of National Defense
- The Swedish Civil Aviation Organization
- Others: U.S. Air Force; airport consultants, etc.

Based on the information procured from these contacts and the personal knowledge of the project team, thirty-six (36) airports in the United States and Canada were short-listed for preliminary interviews. Figure 1 shows a North American map indicating the locations of the various airports. An informal questionnaire was developed to interview the respective airport

Summary of Findings

The findings from the contacts made with various airport authorities and airports are summarized as follows:

- The majority of the airports interviewed to date use potassium acetate as the deicing/anti-icing chemical for their airfields followed by sodium acetate and urea.
- The two widely used chemicals for deicing/anti-icing of the planes are ethylene and propylene glycols.
- Most deicing pads for the aircraft are concrete.
- Of the airports interviewed, Boston Logan International Airport is the only one that indicated that they had some significant distresses detected in the form of stripping of their asphalt concrete pavements. While the exact cause of the stripping was subject to some discussion, it appears to be related to the type and source of asphalt cement and aggregates that were being used at the facility. An extensive research study that included participants from Applied Research Associates and the Western Research Institute resulted in the development of a standard protocol for the evaluation of existing hot mix asphalt pavements at Boston Logan International Airport (*Massachusetts Port Authority, 2005*). The protocol uses a combination of field and laboratory observations and tests to identify the propensity of an asphalt concrete pavement to stripping. Given the observed stripping and extensive use of deicing chemicals at Boston Logan International Airport, it was considered to be a good candidate for the fieldwork portion of this project. Boston airport uses ethylene glycol for pavement deicing which is somewhat unusual.
- Other airports experiencing pavement deterioration but were not sure of the cause include:
 - Aspen/Pitkin County Airport in Colorado
 - Fairbanks International Airport in Alaska
 - Colorado Springs Airport in Colorado
 - Boise Airport in Idaho
 - Freidman Airport in Hailey, Idaho
 - Canadian Forces Base Cold Lake in Alberta and Winnipeg International Airport in Manitoba.

Selection of Airports for Site Investigations and Coring

Final selection of airports for site investigation and pavement coring was based on two factors: (1) the possibility that observed damage was related to the use of DIAICs; and (2) the ability to obtain cores from the pavement in question. Based on these two criteria, the following four airports were selected for site investigation and pavement coring during Phase II of AATP Project 05-03:

- Boston Logan International Airport
- Colorado Springs Airport
- Boise, Idaho Airport
- Freidman Airport, Hailey, Idaho

Descriptions of these airports and the airfield pavements of interest based upon the site investigations are given later in this report.

PRELIMINARY LABORATORY TESTS

In the preliminary test program conducted during Phase I, two aggregates and two binders were evaluated using three procedures:

1. The ultrasonic horn test,
2. FTIR spectroscopy
2. A modification of AASHTO T-283, and
3. A long-term durability test developed specifically for APTP Project 05-03

The two aggregates used were a diabase from Virginia (DB), and a chert gravel from Mississippi (CH). The two binders used were a PG 64-22 and a PG 58-28 supplied by Citgo and both widely used in the Mid-Atlantic States. Two DIAICs were evaluated: potassium acetate (PA) and sodium formate (SF). For all three tests, water was used as a control. The experiment was planned with two mixtures: the diabase aggregate with the PG 64-22 binder, and the chert gravel aggregate with the PG 58-28 binder. As discussed below, WRI tested four mixtures—both aggregates with both binders. AAT initially only performed tests on the diabase/PG 64-22 and chert/PG 58-28 mixtures, but later included the diabase/PG 58-28 and chert/PG 64-22 mixtures in the evaluation of the modified T-283 procedure.

Ultrasonic Horn Test

In the ultrasonic horn test, a compacted briquette-size specimen is placed in a container with de-ionized water and subjected to a partial vacuum. The system is then heated at 60°C for 24 hours after which the tip of the ultrasonic horn is positioned above the specimen and ultrasonic energy is then transmitted to the specimen. Following the ultrasonic conditioning period, the resulting pit depth generated on the briquette surface is used to evaluate moisture susceptibility.

The results of the ultrasonic horn testing are showed in Figure 2. There appears to be a significant difference between the two binders; the PG 64 -22 binder exhibits better resistance compared to the PG 58-28, which is to be expected—stiffer binders often exhibit better moisture resistance in these types of tests. However, there appears to be very little if any significant differences between the two aggregates, or among the two DIAICs and water. The only treatment that showed any significant difference was the potassium acetate with the diabase and PG 64-22 binder, which showed significantly greater pit depth compared to the other mixes treated with potassium acetate.

FTIR Spectroscopy

The four mixtures were analyzed using FTIR spectroscopy, after vacuum saturation with water, and the two DIAIC solutions, and pressure aging for 96 hours at 60°C and 20 atm. Although Finnish researchers had noted the presence of polyaromatic hydrocarbons (PAHs) in HMA subjected to DIAICs, evaluation for PAHs using FTIR was inconclusive, perhaps because the temperatures used during conditioning were not high enough to produce significant amounts of these compounds. However, it was clear that the systems treated with DIAIC solutions did generate significant amounts of carboxylate salts as shown in Figure 3. Specimens aged dry and aged in water showed little or no carboxylate salts in the FTIR analysis. This may be a useful technique for evaluating DIAIC-related damage in HMA pavements. However, as discussed

below, laboratory testing at AAT suggested that only one of the aggregates evaluated in phase I—the Mississippi chert/gravel—is prone to DIAIC-related damage. The FTIR analysis showed generation of carboxylate salts in mixes made with both the chert and the diabase; it appears that the generation of carboxylate salts is the result of a reaction between the DIAICs and the asphalt, and may not be related directly to DIAIC-induced damage.

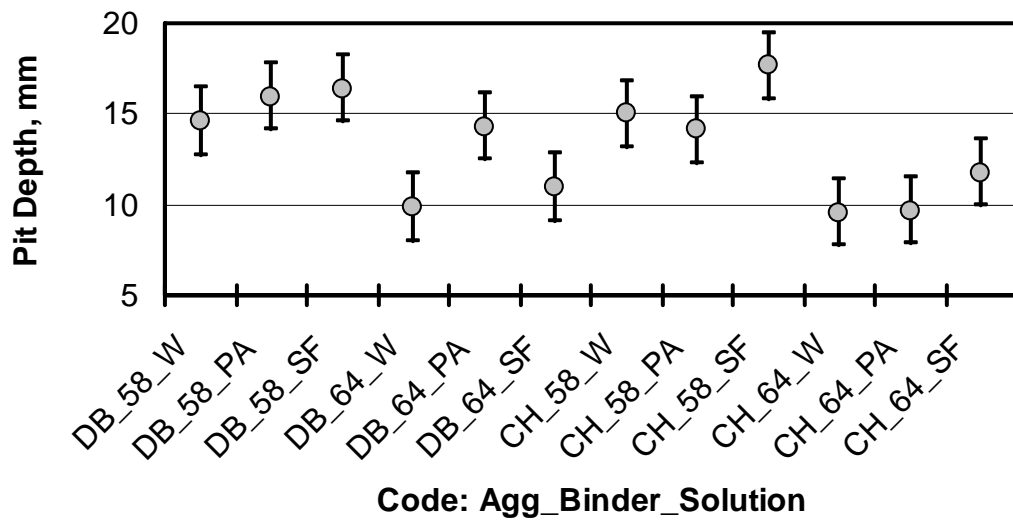


Figure 2. Results of Ultrasonic Horn Test for DIAIC-Induced Damage. Error bars represent D2S precision calculated using a pooled estimate of the standard deviation.

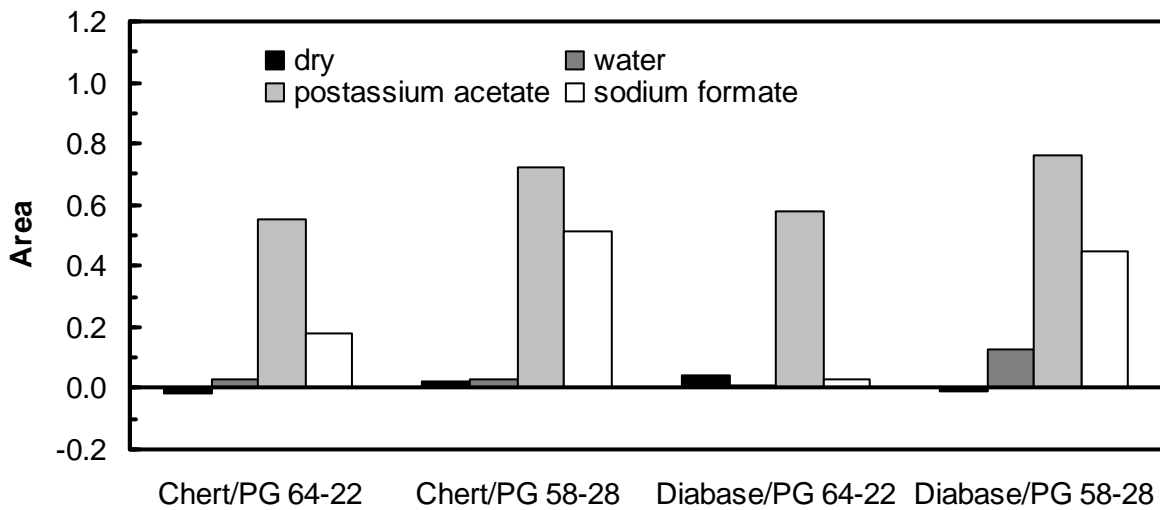


Figure 3. Results of FTIR Testing, Showing Relative Concentration of Carboxylate Salts.

Modified AASHTO T-283 Test

In the modification of the T-283 test, gyratory specimens are vacuum saturated, frozen, and then soaked at 60°C. As discussed previously, three solutions were used—water, 2% PA, and 2% SF. The important comparison in this test is the tensile strength after conditioning in the PA and SF solutions relative to the tensile strength after conditioning in water. The results of these tests are summarized in Figure 4. Initially, only two mixtures were tested using this procedure—the diabase with the PG 64-22 binder, and the chert with the PG 58-28 binder. However, to better evaluate this test method, the remaining two mixtures were tested prior, so that all four combinations of aggregate and binder were tested using this procedure. Both the diabase and chert aggregates appear to be moderately susceptible to moisture damage. The diabase mixtures had a water/dry tensile strength ratio (TSR) of 76 % and 73 % when combined with the PG 64-22 and PG 58-28 binders, respectively. The chert gravel mixtures exhibited TSR values of 81 % and 87 % for the PG 64-22 and PG 58-28 binders, respectively. TSR values above 80 % are generally considered acceptable. The results plotted in Figure 4 show that the tensile strengths for the diabase mixture, when conditioned in the DIAIC solutions, were not significantly different from the strengths when conditioned in water. However, the chert mixtures both show significant reduction in tensile strength when conditioned in DIAIC solutions, compared to their strength when conditioned in water. As should be expected based upon the literature review, the PG 64-22 binder appears to be somewhat more resistant to DIAIC-related damage compared to the PG 58-28 binder. The results of this test are highly significant in that it suggests that the relatively simple IDT test can be used to evaluate the potential for DIAIC-related damage in HMA mixtures, and perhaps more importantly, that susceptibility to DIAIC-related damage is dependent not only upon binder grade and chemistry, but upon aggregate type as well.

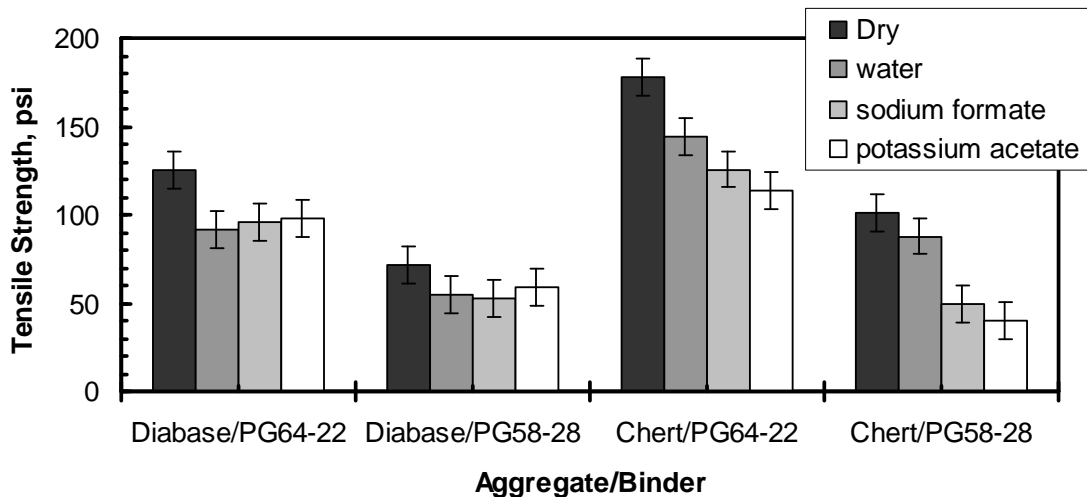


Figure 4. Results of T-283-Based Deicer-Resistance Test. *Error bars represent D2S precision calculated using a pooled estimate of standard deviation.*

Long-Term Durability Test

The long-term durability test was developed by AAT specifically for use in AATTP Project 05-03. This test is designed to produce long-term damage with DIAIC solutions. SPT specimens were prepared following standard procedures. They were conditioned and tested using the following schedule:

1. 72±4 hours in 2 % deicer solution, at 60 ±1 °C.
2. 24±1 hours at -18 °C (specimen wrapped in baggy prior to freezing)
3. 48±2 hours in oven at 50 ±2 °C to dry and condition specimen prior to testing
4. Test for modulus ($|E^*|$) and the repeated load test at 50 °C (100 cycles).
5. Repeat conditioning and testing—once per week for up to 6 weeks.

Because of the short amount of time available for Phase I testing, only the diabase/PG 64-22 and chert/PG 58-28 mixtures were tested using this procedure. Although both mixtures showed some damage as measured by $|E^*|$, the differences in damage measured in this way did not appear to be significant. However, differences in damage measured using the repeated load test were significant; the results of this test are shown in Figures 5 and 6. As with the T-283 based test, for the diabase/PG 64-22 mixture, there is no significant difference in the damage caused by water and the two DIAIC solutions. For the chert/PG 58-28, both the PA and SF solutions caused significantly more damage compared to water. In fact, after two cycles, the chert PG 58-28 mixture disintegrated, which is why data is only shown for one cycle in Figure 6 (testing in the water solution was discontinued after four cycles, since the specimens in both DIAIC solutions had disintegrated after only two cycles). The results of the long-term durability test, using the repeated load test, were in good agreement with the T-283 based test. Taken together, these results strongly suggest that the PA and SF solutions are contributing to moisture damage in mixtures made using the chert gravel aggregate, but not in mixtures made using the diabase aggregate.

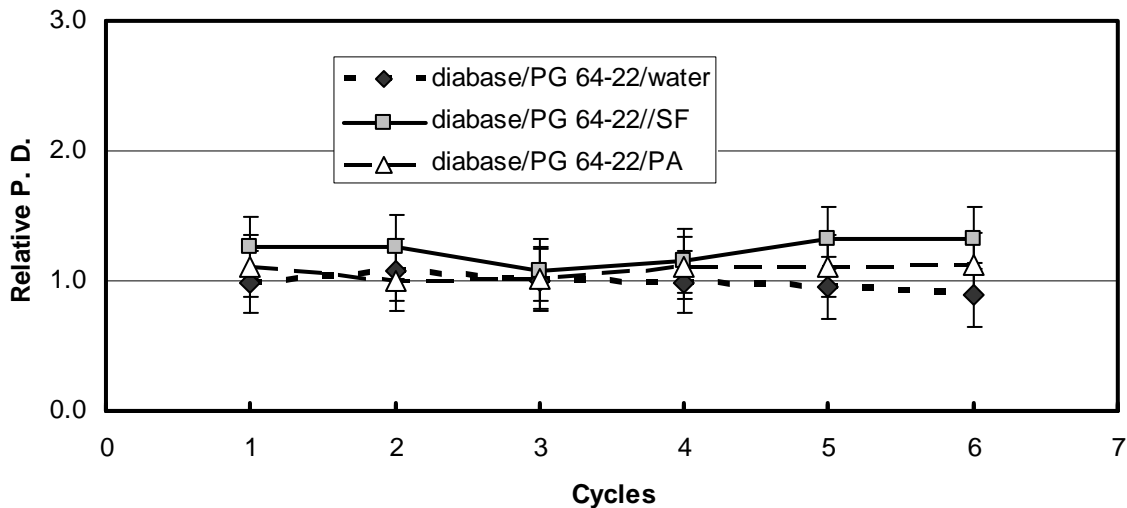


Figure 5. Permanent Deformation Measured During Long-Term Durability Test, Diabase/PG 64-22 Mixture. Error bars represent $D2S$ precision calculated using a pooled estimate of standard deviation.

Aggregate Soundness Testing

AAT also performed soundness tests on both aggregates, using a standard magnesium sulfate solution, and using saturated solutions of PA and SF. The results of this testing are summarized in Table 1. All results show aggregate weight loss values below 15%, a typical allowable maximum value for soundness tests. The loss values for both DIAIC solutions were much lower than for the magnesium sulfate, strongly suggesting that the damage caused to the HMA mixtures was not related to a direct attack on the aggregate by the PA or SF solutions.

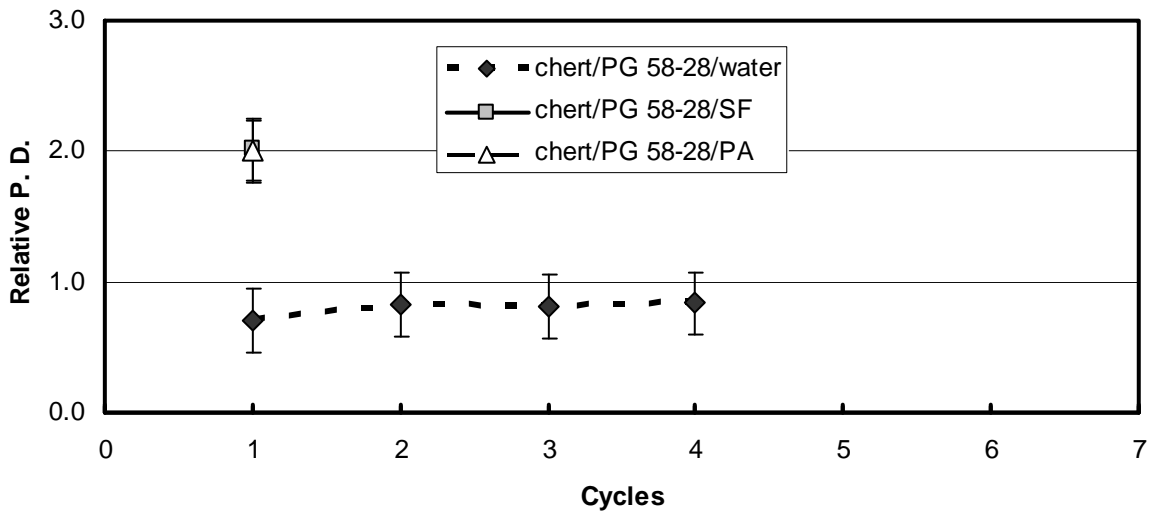


Figure 6. Permanent Deformation Measured During Long-Term Durability Test, Chert/PG 58-28 Mixture. Error bars represent $D2S$ precision calculated using a pooled estimate of standard deviation.

Table 1. Results of Aggregate Soundness Testing.

Aggregate	Fraction	Magnesium Sulfate	Potassium Acetate	Sodium Formate
Diabase	coarse	2.4	0.5	0.5
	fine	7.1	1.1	1.6
Chert	coarse	8.1	1.4	0.9
	fine	10.3	1.9	1.0

Further Refinement of the Immersion Tension Test

Because preliminary testing using the T-283 based procedure appeared to be just as effective in identifying DIAIC-related damage as the much more difficult and time consuming long-term durability test, the research team decided to use the T-283 based procedure for much of the laboratory testing. As discussed below, AAT developed a modified and simplified version of this procedure which allowed a relatively large number of systems to be quickly evaluated in Phase II of Project 05-03. Although the modified AASHTO T-283 procedure was promising for identifying mixtures susceptible to DIAIC-related damage, in its original form—as used in Phase I of Project 05-03—it had several shortcomings:

- Because the T-283 procedure involves vacuum saturation of the specimen prior to conditioning, it is relatively insensitive to air void content and other factors affecting mixture permeability. Because one of the objectives of Project 05-03 was to determine the effect of mixture permeability on any DIAIC-related damage in HMA mixtures, using vacuum saturation in the DIAIC procedure seems to be counterproductive. Vacuum saturation is also time consuming and difficult to perform in a repeatable fashion.
- The T-283 procedure as performed in cold climates normally includes a freezing cycle. However, this study and all previous studies indicate that it is the combination of DIAICs with high temperatures that can lead to HMA damage; the freezing cycle is time consuming and appears unnecessary in the evaluation of DIAIC-related damage in HMA mixtures.
- Testing of a dry, unconditioned mix is not necessary when evaluating DIAIC-related damage in HMA; what is critical is the comparison of tensile strengths of specimens conditioned in water versus the strength of those conditioned in DIAIC solution.
- The repeatability of the AASHTO T-283 procedure is not good, and it is at best only fair in identifying HMA mixtures prone to moisture damage. Many engineers have a low opinion of the usefulness of this test method. Developing a significantly modified procedure addressed specifically to evaluating DIAIC-related damage would help avoid the negative opinions attached to the T-283 procedure, and make implementation of the method easier.

Based upon these considerations, it was determined that the most effective procedure for evaluating DIAIC-related damage would involve soaking laboratory specimens in 2 % DIAC solutions at 60 °C, without vacuum saturation. The test involves evaluation of the IDT strength of the mixture(s) after conditioning in one or more DIAIC, and after conditioning in water as a control. The IDT strength is determined on three replicate specimens, in the same manner as used in AASHTO T-283. Retained tensile strength/deicer treatment (TSR/D) is calculated as tensile strength after DIAIC treatment calculated as a percentage of the strength after treatment in water alone. Retained strength values below 80 % should be considered as evidence of potential DIAIC-related damage for a given HMA/DIAIC system. It is suggested that this procedure be referred to formally as “Immersion Tension Test for Evaluating Deicing/Anti-icing Related Damage in Hot Mix Asphalt.” The abbreviation IT (immersion tension) can be used to refer to the test informally.

A critical question in developing the IT test is how long the specimens should be conditioned in the DIAIC solution (and in water). To determine the answer to this question, specimens of the chert/PG 58-28 mixture were immersed in a 2 % sodium formate solution, and tested for IDT strength after 1, 3, 4 and 5 days immersion. The results are shown in Figure 7. The relative strength determined using the modified T-283 procedure (as discussed above) is also shown in Figure 7. This analysis suggests that for susceptible systems, an immersion time of 4 days should provide damage similar to that which occurs in the modified T-283 procedure. A parallel series of tests were performed in water, and showed little or no damage after 4 days. It is therefore suggested that an immersion time of 4 days be used in the IT procedure.

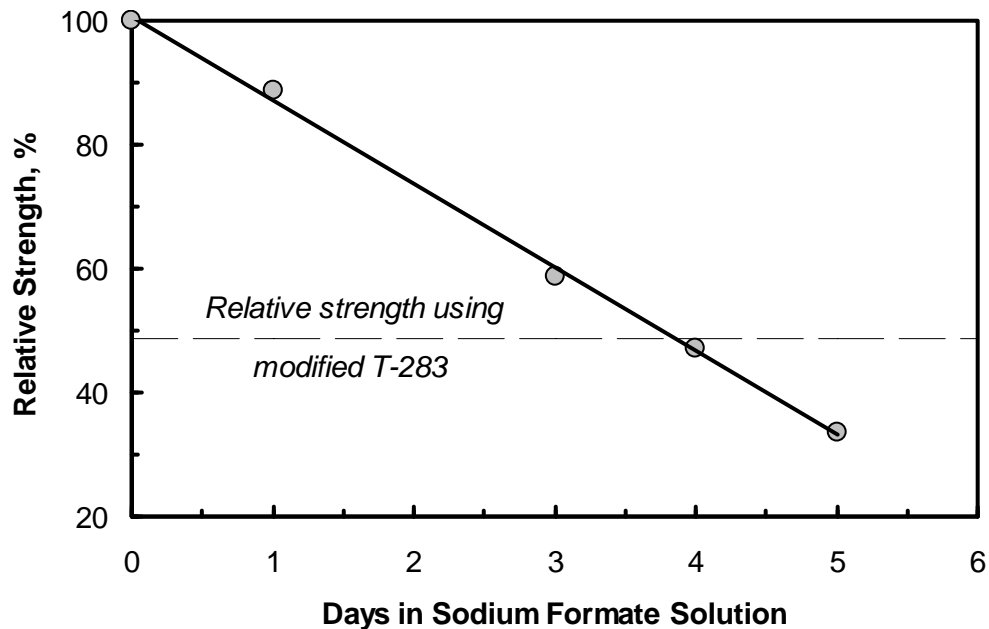


Figure 7. Relative Strength of HMA Immersed in 2 % Sodium Formate Solution (Immersion Tension Procedure).

SUMMARY OF FIELD INVESTIGATIONS ON DIAIC-RELATED DAMAGE IN HMA AIRFIELD PAVEMENTS

After preliminary review of a large number of airports in the U.S. and Canada, only four sites could be found that exhibited possible DIAIC-related damage in HMA pavements. These four airports were the Boise, Idaho, airport; Boston Logan International Airport; Colorado Springs Airport; and the Friedman Memorial Airport in Hailey, Idaho. Summary descriptions for these sites are given below; more detailed descriptions, including numerous photographs, are given in Appendix A. Cores taken from three of these airports (Boise, Boston Logan and Colorado Springs) were eventually subjected to laboratory testing, as described later in this chapter.

Boise, Idaho Airport

The Boise airport (BOI) is a commercial and general aviation airport that is operated by the Boise Department of Aviation and Public Transportation. The current airport has its origins in 1936 and in 1938 had the longest runway in the United States (8,800 feet). A new terminal building was constructed and opened to passenger traffic in 2004. The Boise airport has two asphalt surfaced runways; Runway 10L/28R - 10,000 x 150 feet (3,048 x 46 m), and Runway 10R/28L - 9,763 x 150 feet (2,976 x 46 m).

BOI is an important domestic hub in the mid-northwest of the US for passenger and cargo traffic. The airport experienced unprecedented growth between 1990 and 2000, with passenger numbers doubling to 1.5 million per year. This was due to the number of major employers locating facilities to the area and the consequent expansion in business traffic. Passenger numbers for 2003-04 were almost 3 million per year.

Climate

The Boise area averages about 12 inches of precipitation (rain and liquid equivalent of frozen precipitation), which includes 21 inches of snowfall per year. Temperatures range from an average summer high temperature of 80°F and average winter low temperature of 28°F. The water table at the airport is quite deep at over 55 feet.

Deicing Chemical Use

The airport has typically used urea (common name 46-0-0) and potassium acetate (Cryotech E-36) for pavement deicing. A summary of the pavement deicing chemical usage for the past 7 years is provided below in Table 2.

Table 2. Deicing Chemical Usage at Boise Idaho Airport.

Year	Urea Lbs.	Potassium Acetate Gallons
2000	78,000	1,663
2001	89,000	766
2002	99,000	2,266
2003	65,000	900
2004	117,000	2,250
2005	64,000	266
2006	32,000	1,000

If the ambient temperature in winter is expected to remain above 20°F, the airport typically uses urea for pavement deicing. If temperatures are expected to go below 20°F, then urea is used in combination with the potassium acetate.

Pavement Condition and Recent Issues

Runway 10L/28R was last repaved in 2000 as part of a surface drainage improvement program. A problem was detected over the summer of 2006 in an area of pavement approaching Taxiway E from the west (Figure 8). A depression was observed at this location which has significant traffic load from aircraft banking and turning off the runway. This prompted the closure of the taxiway. An investigation to determine the extent of the pavement deterioration showed severe pavement deterioration beneath the porous friction surface that extended to Runway 10L/28R. After thorough investigation, it was determined that the pavement deterioration was a result of severe stripping of the asphalt mix with almost no evidence of asphalt cement coating on the aggregate from core samples taken from the runway. The stripping was somewhat random, occurred both in and outside of the wheel paths and along the entire length of the runway. Cores taken from the taxiway areas that utilized the same asphalt concrete but were not surfaced with the porous friction course did not show any evidence of stripping.



Figure 8. Boise Runway 10L/28R looking toward the intersection of Taxiway E. *This shows a patched area and an initial depression and shoving noticed in 2006. Aircraft landing on Runway 10 used this taxiway for relatively high speed exits to the terminal area. This resulted in some fairly high braking forces. No additional distress has been observed.*

A review of the pavement condition by Pavement Services, Inc. (February 20, 2007) concluded that the stripping was likely the result of emulsification of the asphalt cement in the presence of water exacerbated by aircraft loading, chemical incompatibility between the asphalt cement and aggregate and the porous friction course trapping water within the pavement structure. It was also suggested that the use of urea as a pavement deicer may also have contributed to the observed stripping problem. It has been suggested that the ammonia derived amine compounds used in liquid anti-stripping additives are similar to the degradation product of urea (ammonia) which in solution could weaken or displace the chemically similar amine bond formed by the liquid antistripping additive between the asphalt cement and aggregate.

Discussions with the local highway district office indicated that they have also used the same aggregate source and asphalt cement and were not experiencing any significant stripping problems.

Given the extent of the deterioration, the runway was rehabilitated in August 2007. All of the affected asphalt concrete was removed and replaced.

Site Visit Observations

Discussions with the airport operations staff indicated that the runway pavements get very hot in the summer. The airport uses pavement sensors to monitor the temperature of the asphalt concrete and indicated that in summer, the temperature of the asphalt concrete at a depth of 12 inches can easily exceed 145°F (maximum temperature recordable by the equipment) by 9:00 am.

The depression and shoving near Taxiway E (Figure 8) that occurred in 2006 has not increased since the closing of Taxiway E. Aircraft landing on Runway 10L typically used Taxiway E as a high speed exit to the terminal area. This would have resulted in some fairly high braking and turning forces in the area of the Taxiway. Since closing this taxiway in 2006, aircraft travel further down the runway to Taxiway D where they are moving more slowly.

There are significant areas of staining on the surface of the pavement. The stains do not coincide with the longitudinal paving joint in the permeable friction course but do appear at regular offsets and may be over joints in the underlying asphalt binder course. The brown coloration of the staining suggest that this might be residual asphalt cement from an emulsion mix of asphalt cement and water being pumped out of the pavement. The cores extracted from the pavement to date show significant deterioration of the asphalt concrete binder course. Additional observations are given in the captions for the photographs taken during the site visit, included in Appendix A.

Asphalt Concrete Sampling

Asphalt concrete cores were extracted from Runway 10L/28R for laboratory testing. One core was extracted from Taxiways C and G on either side of Runway 10L/28R to represent the pavement condition in areas that are not surfaced with the permeable asphalt surface course. Six cores were extracted at various locations along Runway 10L/28R in areas adjacent to cores already extracted that did not show complete deterioration of the asphalt binder course and stripping of the asphalt cement from the aggregate.

Boston Logan International Airport

The Boston Logan International Airport (BOS) has accommodated scheduled air traffic since 1927. The airport is located on 2,400 acres of property directly east of downtown Boston on primarily reclaimed land in the Boston Harbor. Logan airport is one of the 20 busiest airports in the United States with over 400,000 aircraft operations servicing over 27 million passengers per year. The airport has 6 asphalt concrete surfaced runways ranging in length from 2,557 to 10,083 feet.

Climate

The Boston area averages about 42 inches of precipitation (rain and liquid equivalent of frozen precipitation), including 42 inches of snowfall per year. Temperatures are typical of the east coast with an average summer high of 82°F and average winter low of 22°F. The pavements at Logan airport are subject to significant moisture from rainfall, snowfall and fog given its proximity to the Atlantic Ocean.

Deicing Chemical Use

Aircraft deicing and anti-icing has traditionally taken place at the gates at Logan airport. In 2007, a dedicated deicing facility was constructed near the end of Runway 14/32. This deicing facility is intended to permit additional application of deicing chemicals for aircraft that have a long taxi to Runways 4 and 14.

In the past, the airport has typically used a mix of ethylene glycol and urea for pavement deicing. More recently, potassium acetate has been used.

Pavement Condition and Recent Issues

While many of the runway, taxiway and apron pavements at Logan airport are over 50 years old, the majority of the pavement surfaces are less than 20 years old due to resurfacing and rehabilitation treatments. The overall average pavement condition index of the pavements at the airport is 78 which represents very good condition.

Prior to 2004, highway type asphalt concrete mixes were typically specified using AC-20 asphalt cement and ¾ inch top size aggregate. These mixes were prone to rutting and required significant maintenance. In addition, significant stripping of the asphalt concrete was observed in the asphalt concrete mixes placed in the apron areas around the terminal. These mixes contained asphalt cement and a Redicote liquid anti-stripping additive from Bitumar in Montreal, Canada. As a result of the general poor performance of the asphalt concrete, new mixes were designed for the airport which included the use of recycled asphalt concrete, latex modified binders, and 1 percent hydrated lime as an anti-stripping agent.

Logan airport is also very progressive in trying new technologies and has constructed test sections using Citgoflex FR fuel resistant asphalt concrete mixes and warm mix asphalt (Sasobit).

Site Visit Observations

The primary pavements at Logan airport are in good overall condition. Based on the most recent pavement management report, the average pavement condition index (PCI) of the runways is 89 (excellent), taxiways is 80 (very good) and aprons is 73 (very good). The most prevalent distresses observed include low to moderate severity raveling and random cracking. The raveling and cracking is most prevalent in the areas that were overlaid between 2000 and 2002 using the AC-20 from Bitumar with the Redicote liquid anti-stripping additive. The majority of this asphalt concrete was placed on Taxiway K which is the primary access taxiway around the main apron. Additional specific site visit observations are provided in the captions for the photographs included in Appendix A.

Asphalt Concrete Sampling

Asphalt concrete cores were extracted from Taxiway K, and from the Apron in front of the B concourse near Gate 28. The cores were taken to be representative of older pavement (1981) which has been exposed to substantial quantities of deicing chemicals and from the asphalt concrete placed in 2000-2001 that has shown poor performance with significant stripping and raveling.

Colorado Springs Municipal Airport

The Colorado Springs Municipal Airport (COS) is co-located with Peterson Air Force Base approximately six miles southeast of the central business district of the City of Colorado Springs in Colorado. It is the second busiest airport in Colorado. The airport has three primary runways (two surfaced with asphalt concrete and one with concrete). A new terminal was opened in October of 1994. The elevation of the airport is 6,187 ft above sea level.

Climate

The Colorado Springs area averages about 15.5 inches of precipitation (rain and liquid equivalent of frozen precipitation), which includes 37 inches of snowfall per year. Temperatures are typically moderate with January lows on the order of 14°F and July highs in the order of 85°F.

Deicing Chemical Use

Aircraft deicing and anti-icing takes place at the gates at COS airport. Pavement anti-icing and deicing is completed using liquid potassium acetate using spray bar applicators. Airport maintenance personnel also apply sand on some low volume taxiways to improve surface friction during the winter. Potassium acetate is primarily used for Runway 17L/35R and on Taxiways Foxtrot, Golf and Hotel. Deicing chemicals are not generally used for the asphalt concrete surfaced Runway 12/30. Due to the grade of the Runway 30 end area, chemicals used on Taxiway Foxtrot drain onto the end of the runway. Potassium acetate has been used at the airport since 1999.

Pavement Condition and Recent Issues

The majority of the primary airside pavements at COS have exposed concrete surfaces. Runway 17R/35L left was resurfaced in 2003 and is in excellent condition. A GSB 88 fog seal has been applied to the runway surface. The runway surface has been grooved transversely. The only significant surface distress observed was low severity construction joint cracking. Runway 12/30 was last rehabilitated in 1992 and is beginning to show initial signs of distress. Low to moderate severity construction joint cracking with secondary alligator type cracking and raveling is prevalent. Many of the cracks have been sealed using hot-poured rubberized asphalt sealant using over-banding techniques. The airport operations personnel indicated that the Runway 12/30 asphalt concrete included crumb rubber as a modifier. They were unsure of which layers contained the crumb rubber. The runway surface has been grooved transversely.

Site Visit Observations

There appears to be significant ASR type damage to the concrete pavements. Airport operations personnel indicated that it is very difficult to find aggregate in the Colorado Springs area that is not susceptible to ASR. It is manifested by closely spaced parallel cracking at the concrete joints. The Runway 17R/35L pavement is relatively new and did not show any obvious signs of

damage due to deicing chemical use. The cracking, raveling and staining of the asphalt concrete at the Runway 30 end area near Taxiway Foxtrot could be related to the deicing chemicals and an area was selected for coring. Additional specific site visit observations are provided in captions to the photographs included in Appendix A.

Asphalt Concrete Sampling

Asphalt concrete cores were extracted from Runway 30. Cores were taken near the distressed area and an adjacent area showing no distress. Twelve, six-inch diameter cores were extracted from runway 12/30 (south-north) at Colorado Springs airbase. Cores 1-9 were taken from locations with no distresses and 10-12 were taken from locations with alligator cracking and brown residue deposition:

- Cores 1-4 were taken at approximately 80 ft from the start of the runway (most southern point) and just to the west of the runway's center.
- Cores 5-7 taken at approximately 100 ft and just to the west of the runway's center.
- Cores 8-9 taken at about 125 ft and just to the west of the runway's center.
- Cores 10-12 were taken at about 300 ft from the start and just to the east of the runway where longitudinal alligator cracking were present.

Friedman Memorial Airport

Friedman Memorial Airport, formally know as the Sun Valley Airport (SUN) is a commercial and general aviation airport in the town of Hailey in Blaine County, Idaho. The airport has one runway, 6,952 in length.

Climate

The area around the Friedman Airport averages about 16 inches of precipitation (rain and liquid equivalent of frozen precipitation), which includes 81 inches of snowfall per year. Temperatures range from an average summer high temperature of 70°F and average winter low temperature of 12°F.

Pavement Condition and Recent Issues

In 2006, pavement deterioration in the form of cracking, raveling and rutting was observed along the south end of the apron in the primary areas trafficked by aircraft approaching the terminal. In addition, some cracking and deterioration of the asphalt concrete was noticed at the north end of the apron. Pavement cores taken in these areas revealed significant stripping and deterioration of the asphalt concrete.

Runway 13/31 and the airport apron pavement were reconstructed in the spring/summer of 2007. The rehabilitation treatment included milling, expanded asphalt stabilization and an overlay.

Site Visit Observations

A site visit to this airport was not scheduled as the construction work had already been completed and there was nothing of the original pavement to see. Photographs of the pavement, included in Appendix A, were provided by Toothman-Orton Engineering Co. of Boise, Idaho.

Asphalt Concrete Sampling

The airport arranged to have eight cores extracted from the apron pavement prior to the reconstruction work. The cores in general were in very poor condition. Details on the cores are shown in photographs included in Appendix A. Although it was intended that some of these cores be subjected to the IT test, none of the cores were in good enough condition to perform this procedure.

PHASE II LABORATORY TESTING

Phase II laboratory testing included a variety of experiments. The most important of these were tests performed at AAT's Sterling, VA, laboratory using the IT test. Additional work using FTIR analysis was performed at WRI, along with surface tension measurements on the binder/DIAIC systems studied during Phase II laboratory testing. The final activity in Phase II laboratory work was performing the IT test on field cores taken during site visits to a number of airfields to determine if the observed pavement distress was likely caused by DIAIC-related damage.

Immersion Tension Testing

As discussed above, the IT test for evaluating DIAIC-related damage involves soaking laboratory specimens in 2 % DIAC solutions at 60 °C for 4 days, without vacuum saturation. The indirect tensile strength is then determined after conditioning in the selected DIAIC solution, and after conditioning in water as a control. The IDT strength is determined on three replicate specimens, in the same manner as used in AASHTO T-283. TSR/D values (strength after DIAIC treatment as a percentage of strength after water treatment alone) less than 80 % should be considered as evidence of potential DIAIC-related damage for a given HMA/DIAIC system.

Materials

The mixtures used in Phase II testing were composed of five different aggregates and four different binders. The aggregates used included 9.5 mm and 12.5 mm gradations:

- 12.5 mm, fine-graded blend of chert/gravel from Mississippi
- 9.5 mm, coarse-graded blend of diabase from Virginia
- 9.5 mm, dense-graded blend of limestone from Virginia
- 12.5 mm, coarse-graded blend of crushed siliceous gravel from Virginia
- 9.5 mm, coarse-graded blend of greywacke sandstone from Pennsylvania

The chert/gravel and greywacke/sandstone both exhibit a high degree of alkali-silica reactivity. The Virginia gravel exhibited a relatively low level of alkali-silica reactivity. The Mississippi chert/gravel and Virginia diabase aggregates were also used in Phase I of APTP Project 5-3.

Four binders were used in Phase II of AAPT Project 5-3:

- A PG 58-28 binder produced by Citgo
- A PG 64-22 binder also produced by Citgo
- A PG 64-22 binder produced by Lion Oil Company
- A PG 76-22 binder modified with SBS, produced by Citgo

Four DIAICs were included in Phase II testing, all at 2 %:

- Potassium acetate
- Sodium formate
- Sodium acetate
- Propylene glycol

Experiment Design

This phase of the laboratory testing involved three experiments, designed to further evaluate the phenomenon of DIAIC-related damage in HMA pavements. Experiment 1 was designed to evaluate the effect of aggregates on DIAIC-related damage. The objective of Experiment 2 was to evaluate the effect of different binder types and grades on DIAIC-related damage. Experiment 3 was intended to determine if using hydrated lime and/or reducing air void content could help reduce the extent of DIAIC-related damage in susceptible HMA mixtures. The details of these three experiments are given below.

Experiment 1 Design. Aggregate Effects Experiment—this experiment was designed to evaluate the effect of a variety of DIAICs on different aggregates. The study included one binder, five aggregates and four DIAICs. The Citgo PG 64-22 binder was used with five different aggregates: the Mississippi chert/gravel and Virginia diabase aggregates used in Phase I, along with three additional aggregates: an ASR susceptible gravel from Virginia, a limestone aggregate from Virginia, and an ASR susceptible greywacke from the Pennsylvania. The four DIAICs that were used included: propylene glycol, potassium acetate, sodium acetate and sodium formate. All systems were evaluated using the IT procedure described above; therefore all aggregate/binder systems were also evaluated after immersion in water. The results were analyzed graphically and using analysis of variance techniques. Although a more extensive experiment would have been useful, the time and budget limited the number of factors that could be included. An attempt was made to include a range of aggregate types, include several ASR and non-ASR susceptible materials, the most widely used DIAICs, and two of the most commonly used binders in airfield pavements.

Experiment 2 Design. Binder Experiment—in this experiment, the effect of selected DIAICs on additional binders was evaluated using the IT procedure. Three ASR susceptible aggregates: Mississippi chert, Virginia gravel, and Pennsylvania greywacke and two DIAICs: potassium acetate and sodium acetate were selected from the materials used in experiment one, and evaluated using three additional asphalt binders: Citgo PG 58-28, Lyon PG 64-22 and Citgo polymer-modified PG 76-22. As with Experiment 1, the results were analyzed graphically and statistically to determine the effect of binder source and grade on any observed DIAIC-related damage.

Experiment 3 Design. Air Voids and Additive Experiment—in this experiment, the effect of lower air voids, and hydrated lime on DIAIC-related damage was investigated. Three systems (binder/aggregate/DIAIC) were selected from those previously tested: Mississippi chert with Citgo PG 64-22 binder in potassium acetate, Mississippi chert with Lion PG 64-22 in potassium acetate, and Virginia gravel with Citgo PG 58-28 in sodium acetate. Two treatments were evaluated. First, specimens were compacted to 5 % air voids, rather than the standard 7 % used in the Experiments 1 and 2. The second treatment was compaction to 7 % with inclusion of hydrated lime in the HMA as an antistrip additive. As with the other experiments the effect of the DIAICs was evaluated using the IT and appropriate graphical and statistical analyses.

Results of Experiment 1

The results of the Aggregate Effects experiment are summarized in Figure 9 below. Of the five aggregates evaluated, it appears that the only aggregate showing significant susceptibility to DIAIC-related damage is the Mississippi chert/gravel. Because the Pennsylvania greywacke sandstone is known to be highly alkali-silica reactive (ASR) and yet did not exhibit significant damage in this experiment, this suggests that the hypothesis that ASR somehow contributes to DIAIC-related damage may not be correct. The susceptibility of this aggregate—along with the Mississippi chert/gravel and the Virginia gravel—are examined in Experiment 2, discussed below.

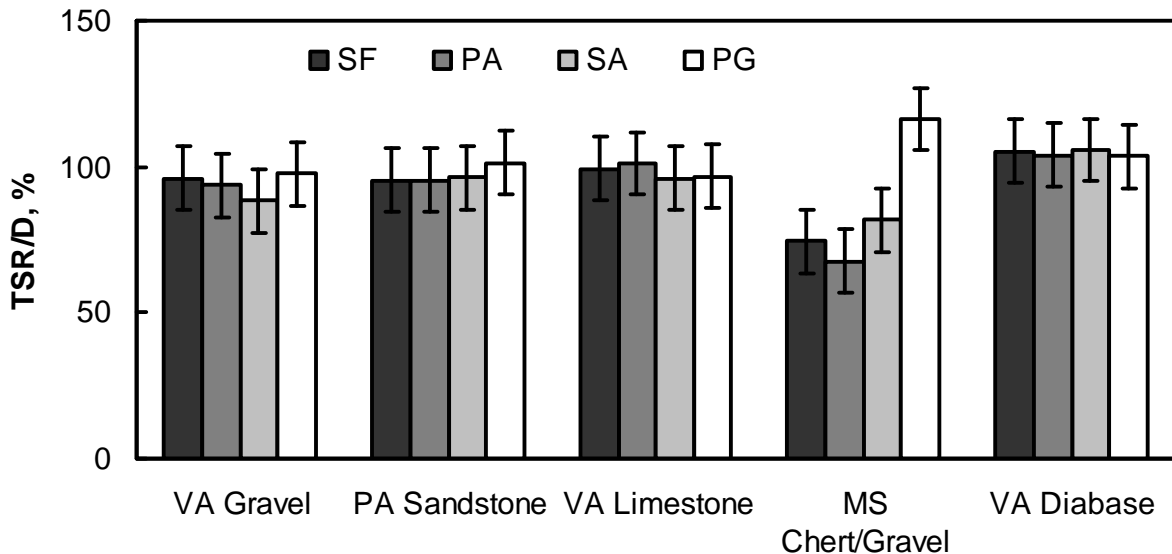


Figure 9. Results of Immersion Tension Testing on Five HMA Mixtures with PG 64-22 Binder, in Solutions of Water and Four Deicing/Anti-icing Chemicals. Error bars show $\pm 2s$ confidence intervals for TSR/D.

Results of Experiment 2

The results of Experiment 2 are summarized in Figure 10. Mixtures made using the Virginia gravel and Pennsylvania sandstone in general showed little damage. There was however a slight

tendency towards greater damage with softer binders. The mixture made using the Virginia gravel and PG 58-28 binder showed some increase in damage with sodium acetate, although it is not clear if the increase is significant (88 % of the tensile strength observed with conditioning in water alone). The damage observed for mixtures made with the Mississippi chert/gravel were in general much greater, and increased with decreasing binder stiffness. This agrees with the findings of European studies that stiffer binders can help minimize DIAIC damage. This also further demonstrates that DIAIC-related damage does not necessarily occur with all siliceous aggregates, or even alkali-reactive aggregates, since the Pennsylvania sandstone is highly reactive and still seems resistant to DIAIC-related damage.

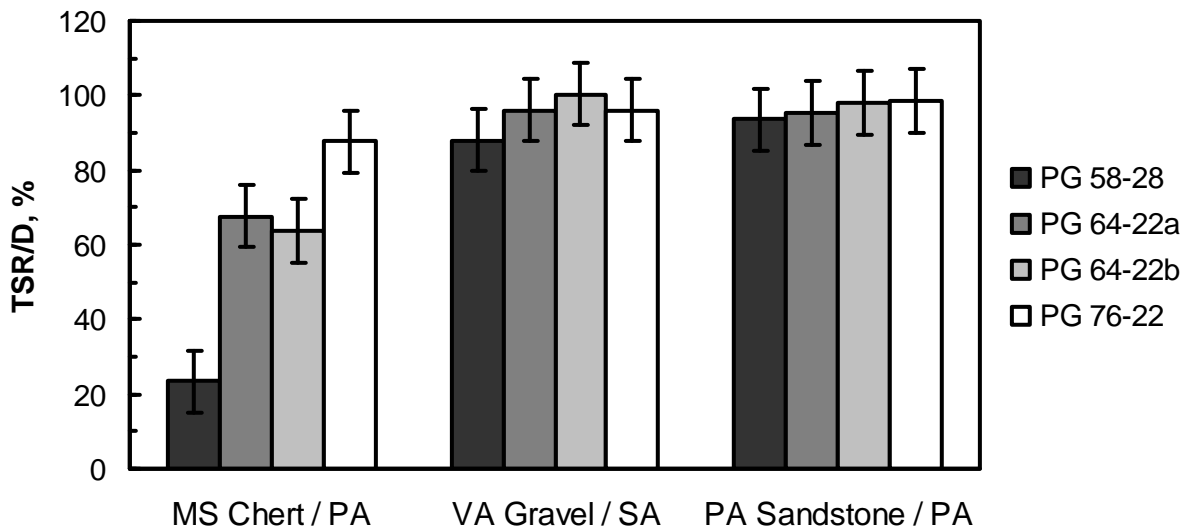


Figure 10. Results of Experiment 2 (Binders). *PA = potassium acetate; SA = sodium acetate.* Error bars show $\pm 2s$ confidence intervals for TSR/D.

Results of Experiment 3

The results of Experiment 3 (air voids/additive) are summarized in Figures 11 through 13 below. In this experiment the effect of two possible treatments for reducing DIAIC-related damage were evaluated. The first was compacting to a lower air void content—5 % rather than 7 %. The second involved treating the mixtures with 1 % hydrated lime. Hydrated lime was used as an additive rather than lithium nitrate as originally planned because the results of Experiments 1 and 2 strongly suggested that the observed DIAIC-related damage is not related to alkali-silica reactivity, but instead is most likely an accelerated form of moisture damage. Figure 11 summarizes the results of testing on the Mississippi chert/gravel/PG 58-28/potassium acetate system. In this case, both improved compaction and hydrated lime significantly reduced the amount of DIAIC-related damage, although the amount of damage measured as the percent reduction in strength relative to the strength of specimens conditioned in water with the identical treatment was still significant—45 and 26 % damage for improved compaction and hydrated lime, respectively. This result does suggest that either approach is promising for reducing DIAIC-related damage. It is possible that both treatments together would provide even better results, although there is the practical question of whether or not it is feasible to require

improved compaction during construction of HMA pavements where the materials are prone to DIAIC-related damage.

Figure 12 summarizes the results of Experiment 3 for the Mississippi chert/gravel/PG 64-22(2)/potassium acetate system. The results for this system are similar to those for the first system, although the benefits of lower air voids and hydrated lime do not appear to be as pronounced. It is likely that the effects of these ameliorative treatments—especially hydrated lime—will vary from system to system. It is also possible that other antistripping additives might prove more effective in reducing DIAIC-related damage, depending on the specific combination of aggregate and binder being used.

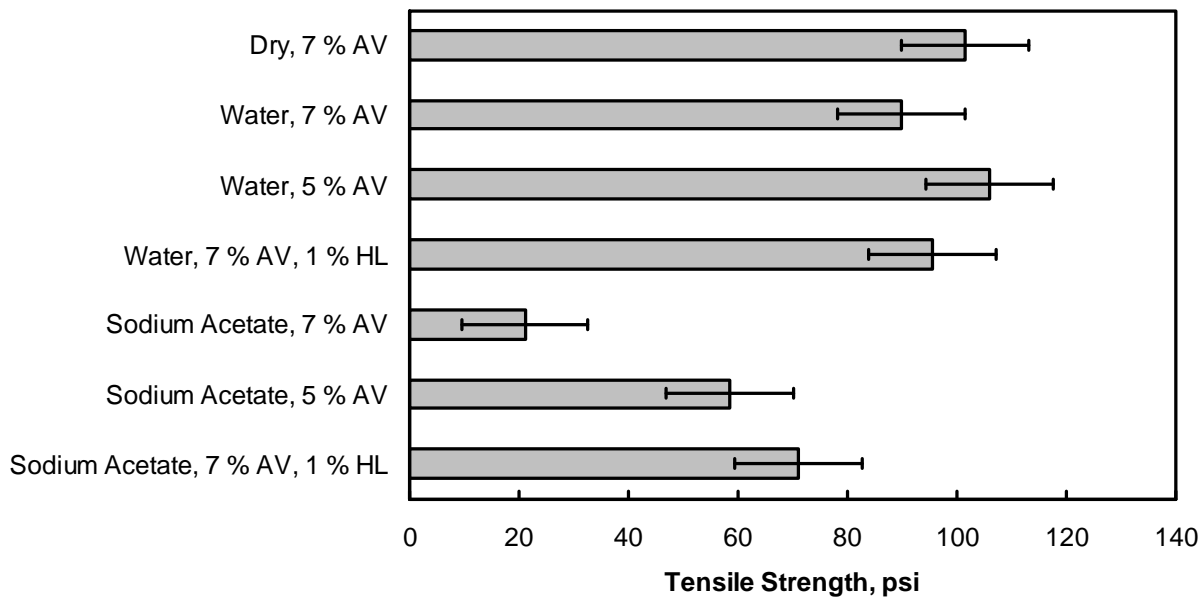


Figure 11. Indirect Tensile Strength Values for Mixture with Mississippi Chert/Gravel and PG 58-28 Binder, Subjected to Different Treatments. Error bars show $\pm 2s$ confidence intervals for tensile strength.

Figure 13 summarizes the results of Experiment 3 for the Virginia gravel/PG 58-28/sodium acetate system. In this case, improving compaction and using hydrated lime both improved tensile strengths, but this increase was nearly the same for specimens conditioned in the sodium acetate solution as it was for specimens conditioned in water alone, suggesting that in this case these treatments had little effect on DIAIC-related damage. However, it must be remembered that the amount of DIAIC-related damage for this system was quite low, perhaps insignificant. This system was included primarily in the interest of including a different aggregate in Experiment 3; it is not clear in this case if there is significant DIAIC-related damage, although the fact that the tensile strength values are consistently slightly lower for specimens conditioned in sodium acetate—regardless of the treatment—indicates that this system does in fact exhibit a small amount of DIAIC-related damage.

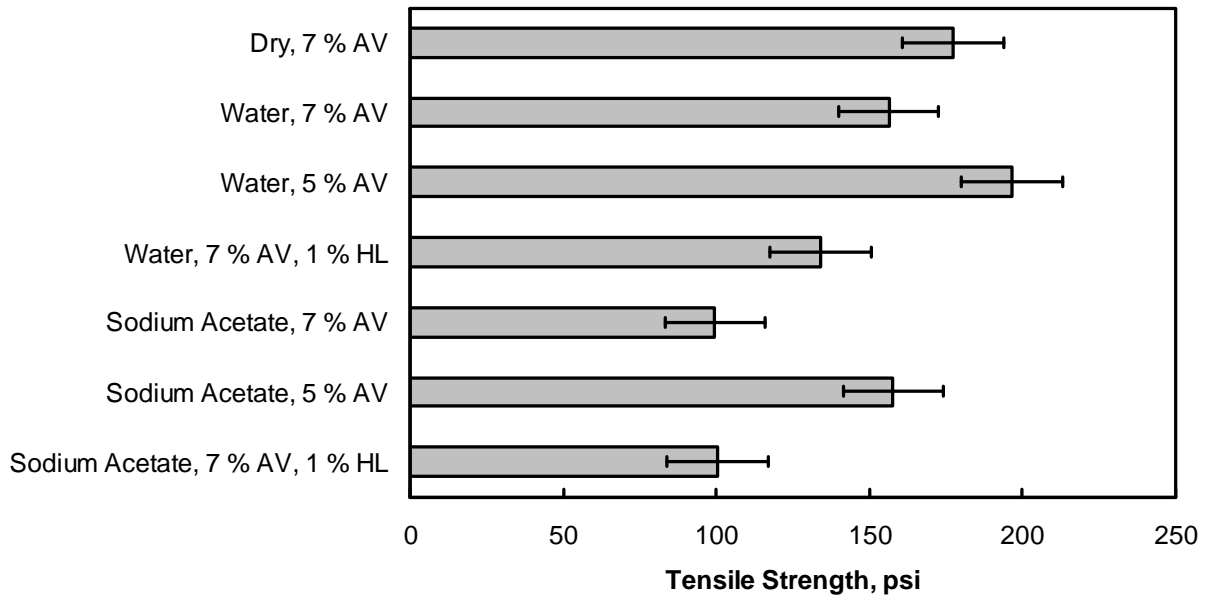


Figure 12. Indirect Tensile Strength Values for Mixtures Made with Mississippi Chert/Gravel and PG 64-22(2) Binder, Subjected to Different Treatments. *Error bars show $\pm 2s$ confidence intervals for tensile strength.*

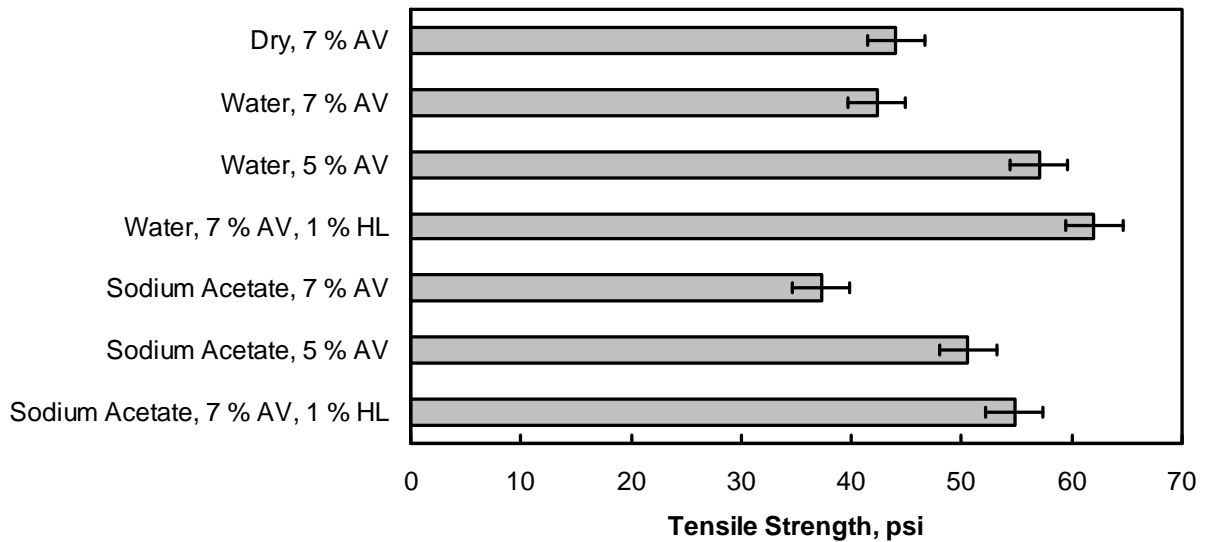


Figure 13. Indirect Tensile Strength Values for Mixture Made with Virginia Gravel and PG 58-28 Binder, Subjected to Different Treatments. *Error bars show $\pm 2s$ confidence intervals for tensile strength.*

FTIR Analyses

Details of the results of FTIR analyses on the asphalt binder/DIAIC solutions are given in Appendix B. As discussed above, it was initially surmised that FTIR analysis, by identifying carboxylate salts, might prove to be a useful “flag” indicating DIAIC-related damage in HMA pavements. However, the Phase II testing indicated that the carboxylate salts observed in some asphalt samples subjected to DIAIC-solutions were not in fact the result of any reaction with the DIAIC-solutions. Other problems were encountered in the FTIR analyses, as discussed in Appendix B.

Surface Tension Measurements

Nuclear Magnetic Resonance (NMR) imaging methods can be used to obtain asphalt-water interfacial parameters, such as contact angles and surface tensions. These data can then be used to develop a better understanding of asphalt/water interactions and an understanding of what parameters are important for determining the moisture sensitivity of asphalts. The results of this study indicate that deicing chemicals have an affect on the interfacial properties of asphalt and water. In addition, the added affect of increased density of some concentrated aqueous solutions of DIAICs could have a marked affect on the penetration of water into asphalt.

The asphalt samples used in this study were the PG 58-28 binder and the PG 64-22 binder used in Experiment 1 (Aggregate Effects Experiment). The deicer solutions prepared for NMR imaging measurements are listed in Table 3; details of solution preparation are given in Appendix C of this report. The density and DuNuoy surface tension of each deicer solution (DuNuoy ring tensiometer) are also included in Table 3. The density of each solution was determined by weight comparison of the deicer solution versus an equal volume of distilled water.

Table 3. Deicer Samples Used in the NMR Imaging Measurements.

Code	Concentration	DuNuoy Surface	
		Tension Dynes/cm	Density g/ml
Water	Water	72.4	1.000
2% SF	2% sodium formate	69.1	1.0144
2% PA	2% potassium acetate	71.3	1.0084
2% PG	2% propylene glycol	68.0	1.0096
35% PG	35 % propylene glycol	51.5	1.0359
35% SF	35% sodium formate	52.6	1.2456
35% PA	35% potassium acetate	65.3	1.1945

Contact angles between asphalt, water and deicer solution were determined through NMR imaging of water drops on asphalt surfaces; these experiments were carried out at a nominal proton resonance frequency of 200 MHz using a Varian/Chemagnetics microimaging probe. Images of each water drop on asphalt were scanned, then transferred to a laboratory computer, and enhanced using Image-Pro Plus 6.2 (2007 Media Cybernetics). An example of enhanced images is shown in Figure 14 for the PG 58-28 binder in a 2 % solution of propylene glycol. The

contact angles for the asphalt-water and asphalt-deicer solution interfaces were then measured using the Drop Snake, drop-shape analysis plug-in for ImageJ software from the National Institutes of Health, USA. Contact angles measured on the left and right sides of the water drops are given in Table 4. Details of the procedure are included in Appendix C.

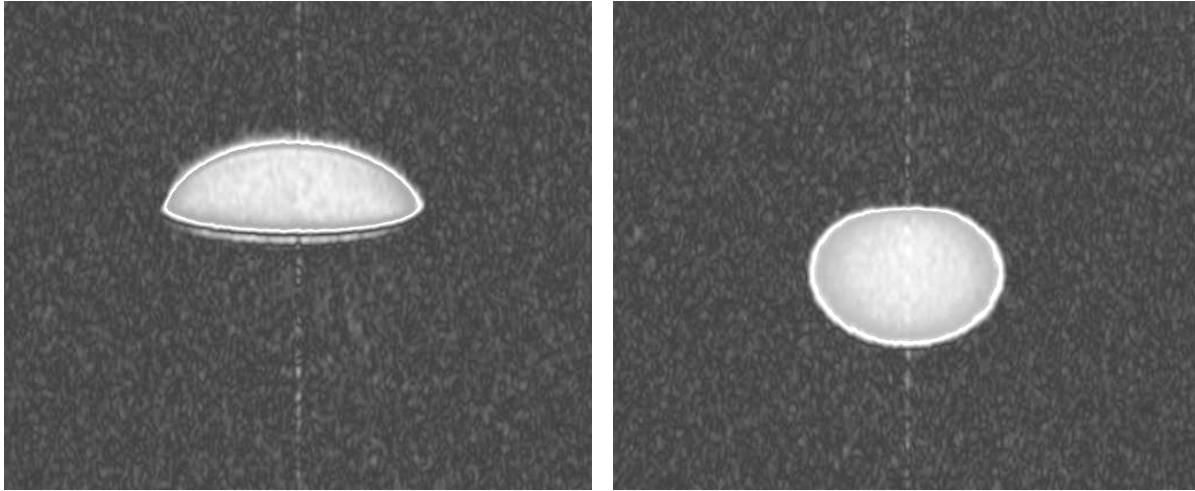


Figure 14. NMR images of the PG 58-28 binder in 35 % potassium acetate at 0h and 22h illustrating contrast enhancement.

Table 4. Contact Angles Measured at the Left and Right Sides of the Water Drop Images.^a

Solution	Left contact angles				Right contact angles			
	$\beta(t_1)$	$\alpha(t_1)$	$\beta(t_2)$	$\alpha(t_2)$	$\beta(t_1)$	$\alpha(t_1)$	$\beta(t_2)$	$\alpha(t_2)$
<i>PG 58-28 Binder</i>								
Water	70	33	0	146	67	49	0	150
2% SF	73	39	0	150	72	46	0	144
2% PA	63	35	0	144	57	49	0	142
2% PG	68	36	0	152	69	45	0	146
35% PG	52	31	0	158	53	30	0	146
35% SF	46	25	0	151	45	29	0	149
35 % PA	68	42	0	132	63	44	0	135
<i>PG 64-22 Binder</i>								
Water	76	16	0	137	83	21	0	155
2% SF	71	22	0	161	72	39	0	155
2% PA	74	20	0	165	73	33	0	147
2% PG	77	18	0	143	73	35	0	149
35% PG	62	22	0	150	73	27	0	134
35% SF	68	21	5	136	78	25	5	142
35 % PA	84	22	9	153	87	34	9	141

^a β refers to the angle above the asphalt surface; α refers to the angle below the asphalt surface, t_1 and t_2 refer to times at which the images were recorded

After measuring the surface tensions of the solution used for the drop by another method (DuNuoy balance, Wilhelmy plate, etc.) and measuring the contact angles of the droplets on the asphalt by NMR imaging, the asphalt-droplet surface tension can be calculated from NMR measurements made on a droplet of water as it rests on—and eventually passes down through—a specimen of asphalt binder. A description of the theory and mathematics involved in this calculation are given in Appendix C, along with a variety of images from this experiment. The values of the surface tensions calculated from the contact angle measurements are given in Table 5. Surface tensions were calculated from contact angles measured on the left and right sides of the images, as well as from the average value of the asphalt-air surface tension. For several samples there were considerable differences in the left and right contact angles; however the agreement among the calculated surface tensions was quite good, particularly for the PG 58-28 sample set. This can be attributed to the surface tensions being determined using two contact angles at the point of contact, and even if the left and right angles are different, the fact that the surface tensions are calculated using two angles, which partially compensates for these differences. Additional analysis including calculation of other parameters related to contact angles and surface tension are given in Appendix C.

Table 5. Surface Tensions Calculated from NMR Contact Angle Measurements.^{a,b}

Solution	Left Contact Angle		Right Contact Angle		Average	
	Asphalt-H ₂ O ^c	Asphalt-air ^c	Asphalt-H ₂ O ^c	Asphalt-air ^c	Asphalt-H ₂ O ^c	Asphalt-air ^c
<i>PG 58-28 Binder</i>						
Water	29 (25)	49	29 (27)	47	29 (25)	48
2% SF	30 (33)	43	32 (35)	43	31 (33)	43
2% PA	22 (17)	52	20 (11)	54	21 (17)	53
2% PG	26 (25)	46	29 (31)	44	27 (25)	45
35% PG	11 (17)	41	12 (17)	41	12 (17)	41
35% SF	9 (10)	45	9 (10)	45	9 (10)	45
35 % PA	30 (29)	46	26 (23)	48	28 (29)	47
	Average	46		46		46
	Std. dev	4		4		4
<i>PG 64-22 Binder</i>						
Water	33 (25)	49	35 (33)	41	34 (24)	45
2% SF	25 (21)	46	29 (24)	43	27 (20)	44
2% PA	24 (24)	45	27 (23)	46	26 (23)	46
2% PG	31 (28)	44	29 (25)	43	30 (27)	44
35% PG	15 (19)	38	23 (28)	36	19 (18)	37
35% SF	20 (24)	38	25 (22)	33	23 (23)	36
35 % PA	33 (38)	36	39 (44)	35	36 (37)	36
	Average	42		40		41
	Std. dev	5		5		5

^a Drop Snake, left images, 5/21/08; ^b numbers in () are calculated using the average asphalt-air surface energy, ^c all data in Dynes/cm.

Previous NMR imaging measurements of the SHRP core asphalts gave values in the range of 26 to 38 dynes/cm for the asphalt-water surface tension and 38 to 48 dynes/cm for the asphalt-air surface tensions (Miknis et al. 2005a, b). The asphalt-water surface tensions of the deicer liquids are somewhat less than those of the asphalt-water surface tensions determined for the SHRP asphalts (Miknis et al. 2005a,b), but still of the same magnitude. The asphalt-water surface tension values for the propylene glycol and sodium formate solutions in both asphalts (particularly the PG 58-28 binder) are lower than the others in the sample sets.

Field Tests

The results of the IT tests on the field cores are summarized in Figure 15. This plot shows average IDT strength for field cores from the three airfields, including both new and old pavements at the Boston Logan airport. Because there was significant variation in bulk specific gravity within the various groups of cores, the strengths have been adjusted for variation in bulk specific gravity, based upon the results of an analysis of variance (discussed in the following paragraph). There are differences in the IT strengths among the airfields—strengths for the cores from the old Boston Logan pavement are especially high. In general, the potassium acetate treatment appears to slightly increase the IT strength. This suggests that none of these HMA mixtures are susceptible to DIAIC-related damage.

An analysis of variance was performed on the IT data collected on the field cores. Factors included project (airfield), treatment (water/potassium acetate), and the interaction of these two factors. Both project and treatment were considered as fixed effects. Relative bulk specific gravity was included as a covariate. Relative bulk specific gravity was calculated as the bulk specific gravity for a core divided by the average bulk specific gravity for all cores for that project. Inclusion of this covariate reduced the overall error in the model and improved the statistical power of the resulting tests. The results of the analysis of variance are summarized in Table 6. Project was a significant factor, but treatment and the interaction of project and treatment were both statistically insignificant—therefore, as noted above, there appears to be no DIAIC-related damage in any of these pavements.

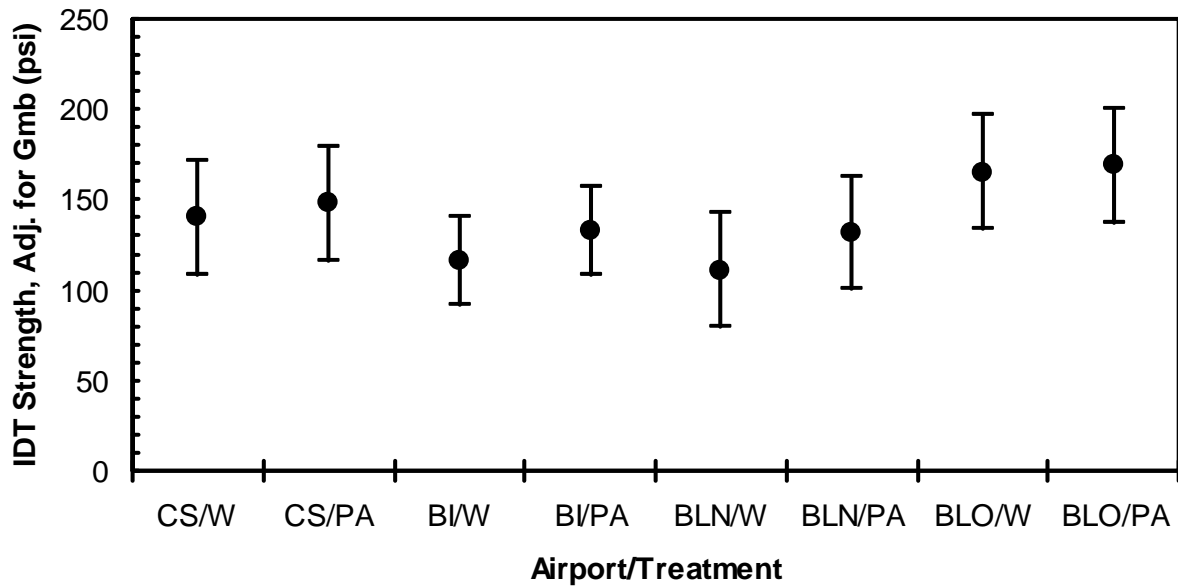


Figure 15. Results of IT Test for HMA Cores Taken at Four Airfields: Colorado Springs (CS); Boise, Idaho (BI); Boston Logan/New (BLN); and Boston Logan/Old (BLO). *W* = water, *PA* = potassium acetate solution. Error bars represent two standard-deviation confidence limits for the average strength.

Table 6. Results of Analysis of Variance of Field Test Data.

Source	Degrees of Freedom	Adjusted Sum of Squares	Adjusted Mean Squares	F	P
Relative Bulk Spec. Gravity	1	7,575.2	7,575.2	10.18	0.005
Project	3	8,857.5	2,952.5	3.97	0.024
Treatments	1	957.2	957.2	1.29	0.271
Project × Treatment	3	268.5	89.5	0.12	0.947
Error	19	14,138.1	744.1		
Total	27				

Covariate Statistics

Term	Coefficient	Standard Deviation	T	P
Constant	-939.9	338.3	-2.78	0.012
Relative Bulk Spec. Gravity	1079.0	338.3	3.19	0.005

CHAPTER 3

DISCUSSION OF RESULTS

Previous research on DIAIC-related damage in HMA airfield pavements has suggested that the primary cause of this distress is essentially moisture damage accelerated by the lower surface tension and relatively high density of DIAIC-solutions compared to water. The lower surface tension of the DIAIC-solutions allows them to more thoroughly wet asphalt binder surfaces in a mixture, and also allows more rapid penetration at the asphalt-aggregate interface. The higher density of DIAIC-solutions means that these will penetrate into the HMA pavement more rapidly than water, simply because of their greater density.

After Phase I of APTP Project 5-3, it was believed that a useful alternative hypothesis to DIAIC-related damage in HMA airfield pavements was that the alkalis in DIAIC solutions was reacting with susceptible aggregates, causing an expansive reaction at the asphalt-aggregate interface and a resulting loss of bond strength and eventual disintegration of the mixture. This was based upon the high degree of susceptibility to DIAIC-related damage observed in the Mississippi chert/gravel aggregate, which exhibits a high degree of alkali-silica reactivity (ASR). To further investigate this hypothesis, Phase II testing included aggregates exhibiting a range of ASR reactivity, including a Pennsylvania greywacke/sandstone that is very reactive.

Phase II testing did not however support the hypothesis that DIAIC-related damage is related to alkali-silica reactivity. None of the additional aggregates tested, including those that were highly reactive, showed significant DIAIC-related damage. Apparently, the Mississippi chert/gravel is unusual in its susceptibility to this type of damage. It is concluded on the basis of research conducted during APTP Project 5-3 that the findings made by earlier research are correct, in that DIAIC-related damage is in essence moisture damage, accelerated by the relatively low surface tension of most DIAIC solutions, and the high density of some concentrated DIAIC solutions. Furthermore, it appears that this type of distress is relatively rare in the U.S. and Canada.

In cases where DIAIC-related damage is a problem, solutions are relatively straightforward. It should for the most part be treated as a type of moisture induced damage. Changing binders and/or aggregate may reduce or even eliminate the problem. If this is not economical, anti-strip additives might reduce the susceptibility to DIAIC-related damage to an acceptable level. Using harder binders—especially PG 76-22 polymer modified binders—tends to greatly reduce the extent of DIAIC-related damage.

The IT test developed during APTP Project 5-3 is potentially a useful tool for identifying HMA mixtures susceptible to DIAIC-related damage. It is simple, quick and can be run by any laboratory familiar with AASHTO T-283 and related procedures. It was effective in identifying one aggregate—the Mississippi chert/gravel—as producing HMA mixtures prone to DIAIC-related damage. However, it cannot be concluded that the test is highly effective in identifying mixes prone to HMA damage. This is because it was not possible during this study to locate a significant number of HMA mixtures prone to DIAIC-related damage that could then be subjected to the IT procedure to determine if this test would correctly identify their poor

resistance to deicer-related damage. Cores from airfield pavements tested during this study represented material that was possibly subject to DIAIC-related damage—it was not certain that these cores represented HMA prone to this type of distress. Therefore, prior to full implementation of this test procedure, additional evaluation of its effectiveness in identifying DIAIC-related damage is warranted. Such an evaluation should ideally consist of testing a number of different HMA mixtures, including a number known to be susceptible to DIAIC-related damage. Such samples could possibly be procured from laboratories in Finland and other parts of Scandinavia where this problem is apparently more widespread. Appendix D is a draft specification for the IT test given in ASTM format.

FTIR analysis does not appear to be useful in testing and analyzing HMA mixtures to determine either susceptibility to DIAIC-related damage, or to determine if such damage has occurred in cores taken from airfield pavements. Although carboxylate salts were found in many specimens tested as part of APTP Project 5-3, it was determined that these were not the result of reactions between deicing chemicals and carboxylic acids in the asphalt binders. Surface tension tests did indicate that many DIAIC solutions have reduced surface tension values compared to water. This supports the hypothesis proposed by Finnish researchers that the lower surface tension and relatively high density of DIAIC solutions tends to increase the severity of moisture damage in susceptible systems.

Although Finnish researchers have suggested that poly-aromatic hydrocarbons can be formed in HMA mixtures subjected to DIAICs followed by high temperatures, there was no conclusive evidence in APTP Project 5-3 that generation of PAHs is a problem in pavements subjected to DIAICs. The discrepancy in these findings might be due to the somewhat lower temperatures used in APTP Project 5-3 compared to those occurring in some of the test procedures used in the Finnish studies.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

The conclusions listed below are based in part upon the literature review conducted during Phase I of the project, and also upon the results of laboratory testing performed during Project 05-03. The most important of these laboratory tests involved the immersion tension (IT) test, a procedure similar to AASHTO T 283 which appears to be very promising in identifying deicer/anti-icing chemical-related damage in HMA pavements. The IT test procedure was used to investigate the effects of various factors on DIAIC-related damage: DIAIC type, aggregate type, binder grade, air void content, and the addition of hydrated lime to the mixture. In most cases, the results of these laboratory tests agreed with findings made by other researchers as summarized in the literature review. Work performed as part of AAPT Project 05-03 has resulted in 12 important findings:

1. Certain deicer/anti-icing chemicals (DIAICs) appear to increase the extent of moisture-induced damage in some HMA mixtures. This should not be considered a unique form of distress, but an accelerated type of moisture damage. For this reason it should be called DIAIC-related damage.
2. DIAIC-related damage in HMA mixtures does not appear to be common.
3. DIAIC-related damage appears to be limited to HMA mixtures containing significant amounts of siliceous aggregate. However, many siliceous aggregates may not exhibit significant DIAIC-related damage.
4. Relatively high temperatures are required for acetate- and formate-based DIAICs to cause damage in HMA mixtures. Therefore, laboratory tests for identifying such distress should subject the HMA being studied to minimum temperatures of about 60°C in order to promote DIAIC-related damage. Review of the literature suggests that tests performed at lower temperatures are not useful in identifying HMA mixtures prone to DIAIC-related damage.
5. DIAIC-related damage is generally more severe for HMA made with softer binders. Conversely, using stiffer binders, especially PG 76-22 polymer modified binders, will tend to minimize DIAIC-related damage in susceptible mixes.
6. DIAIC-related damage increases in severity with increasing in-place air void content. Therefore, when an HMA is suspected of being susceptible to DIAIC-related damage special care should be taken to ensure that pavements constructed with the mixture are properly compacted.
7. The addition of hydrated lime may decrease the severity of DIAIC-related damage in susceptible mixes. Although not evaluated as part of this study, it is possible that other anti-stripping additives might also be effective in reducing the extent of DIAIC-related damage.
8. The IT test is promising as a relatively simple and effective means for identifying HMA mixtures susceptible to damage from acetate- and formate-based DIAICs. This procedure involves conditioning gyratory specimens in DIAIC solution at

60°C for four days, and then measuring the indirect tensile strength. This strength is compared to that of specimens conditioned in water at 60°C for four days. A loss of strength of 20 % or more when conditioned in DIAIC solution (compared to water) should be considered an indication that the mixture is susceptible to DIAIC-related damage.

9. Because DIAIC-related damage is not common, it might be difficult to further evaluate the reliability of the IT test in identifying DIAIC-related damage. Therefore, identification of DIAIC-related damage should rely largely on engineering judgment and experience. DIAIC-related damage in a flexible pavement is suggested by the following characteristics: (1) unusually severe moisture-induced damage; (2) significant use of DIAICs; and (3) the presence of significant amounts of siliceous aggregate in the HMA mixture. If all three characteristics are present, DIAIC-related damage should be suspected. If acetate- and/or formate-based DIAICs have been used on the pavement, the IT test may be performed to confirm that these chemicals are contributing significantly to moisture-induced damage in the pavement. The IT test has not been evaluated in identifying damage caused by exposure of HMA pavements to urea solutions.
10. During this study, Fourier transform infrared spectroscopy (FTIR) showed significant amounts of carboxylate salts generated during conditioning of HMA specimens in DIAIC solutions. Unfortunately, large amounts of such compounds were generated both in mixtures exhibiting DIAIC-related damage, and in those not exhibiting DIAIC-related damage. Therefore, the FTIR test does not appear to be useful as a flag for identifying DIAIC-related damage. The presence of carboxylate salts does however suggest that neutralization of carboxylic acids is one of the mechanisms for DIAIC-related damage.
11. There was no conclusive evidence in APTP Project 5-3 that poly-aromatic hydrocarbons are generated in significant amounts in HMA pavements subjected to DIAICs. However, it is possible that these compounds are produced in HMAs subjected to DIAICs followed by very high temperatures—temperatures significantly exceeding the 60°C conditioning used in this study.
12. There are several areas of research still needed to better understand and prevent DIAIC-related damage in HMA pavements. Further effort is needed in evaluating the IT test procedure for a wider range of aggregates and asphalt binders, to confirm that this test is in fact effective in identifying mixtures susceptible to DIAIC-related damage. Work is also needed in further evaluating the effectiveness of different anti-stripping additives in reducing DIAIC-related damage. However, it may be difficult to justify such research because of the apparent rarity of DIAIC-related damage in HMA pavements. Further field investigations might help identify additional incidences of DIAIC-related damage. Such field work should broadly investigate incidences of moisture-induced damage in airfield pavements made using siliceous aggregates; it is quite possible that such damage might in some cases be partly the result of DIAIC-related damage.

CHAPTER 5

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**APPENDIX A: FIELD INVESTIGATION OF DIAIC-RELATED
DAMAGE IN HMA AIRFIELD PAVEMENTS**

After preliminary investigation of a large number of airports in the U.S. and Canada, only four sites could be found that exhibited possible DIAIC-related damage in HMA pavements. These four airports were the Boise, Idaho, airport; Boston Logan International Airport; Colorado Springs Airport; and Hailey-Friedman Airport (Boise, Idaho). Detailed reports for these for sites are given below.

Boise, Idaho Airport

The Boise airport (BOI) is a commercial and general aviation airport that is operated by the Boise Department of Aviation and Public Transportation. The current airport has its origins in 1936 and in 1938 had the longest runway in the United States (8,800 feet). A new terminal building was constructed and opened to passenger traffic in 2004. The Boise airport has two asphalt surfaced runways; Runway 10L/28R - 10,000 x 150 feet (3,048 x 46 m), and Runway 10R/28L - 9,763 x 150 feet (2,976 x 46 m).

BOI is an important domestic hub in the mid-northwest of the US for passenger and cargo traffic. The airport experienced unprecedented growth between 1990 and 2000, with passenger numbers doubling to 1.5 million per year. This was due to the number of major employers locating facilities to the area and the consequent expansion in business traffic. Passenger numbers for 2003-04 were almost 3 million per year.

Climate

The Boise area averages about 12 inches of precipitation (rain and liquid equivalent of frozen precipitation), which includes 21 inches of snowfall per year. Temperatures range from an average summer high temperature of 80°F and average winter low temperature of 28°F. The water table at the airport is quite deep at over 55 feet.

Deicing Chemical Use

The airport has typically used urea (common name 46-0-0) and potassium acetate (Cryotech E-36) for pavement deicing. A summary of the pavement deicing chemical usage for the past 7 years is provided below in Table B-1.

Table B-1. Deicing Chemical Usage at Boise Idaho Airport.

Year	Urea <i>Lbs.</i>	Potassium Acetate <i>Gallons</i>
2000	78,000	1,663
2001	89,000	766
2002	99,000	2,266
2003	65,000	900
2004	117,000	2,250
2005	64,000	266
2006	32,000	1,000

If the ambient temperature in winter is expected to remain above 20°F, the airport typically uses urea for pavement deicing. If temperatures are expected to go below 20°F, then urea is used in combination with the potassium acetate.

Pavement Condition and Recent Issues

Runway 10L/28R was last repaved in 2000 as part of a surface drainage improvement program. A problem was detected over the summer of 2006 in an area of pavement approaching Taxiway E from the west (Figure 8). A depression was observed at this location which has significant traffic load from aircraft banking and turning off the runway. This prompted the closure of the taxiway. An investigation to determine the extent of the pavement deterioration showed severe pavement deterioration beneath the porous friction surface that extended to Runway 10L/28R. After thorough investigation, it was determined that the pavement deterioration was a result of severe stripping of the asphalt mix with almost no evidence of asphalt cement coating on the aggregate from core samples taken from the runway. The stripping was somewhat random, occurred both in and outside of the wheel paths and along the entire length of the runway. Cores taken from the taxiway areas that utilized the same asphalt concrete but were not surfaced with the porous friction course did not show any evidence of stripping.

A review of the pavement condition by Pavement Services, Inc. (February 20, 2007) concluded that the stripping was likely the result of emulsification of the asphalt cement in the presence of water exacerbated by aircraft loading, chemical incompatibility between the asphalt cement and aggregate and the porous friction course trapping water within the pavement structure. It was also suggested that the use of urea as a pavement deicer may also have contributed to the observed stripping problem. It has been suggested that the ammonia derived amine compounds used in liquid anti-stripping additives are similar to the degradation product of urea (ammonia) which in solution could weaken or displace the chemically similar amine bond formed by the liquid antistripping additive between the asphalt cement and aggregate.

Discussions with the local highway district office indicated that they have also used the same aggregate source and asphalt cement and were not experiencing any significant stripping problems.

Given the extent of the deterioration, the runway has been rehabilitated in August 2007. All of the affected asphalt concrete was removed and replaced.

Site Visit Observations

Discussions with the airport operations staff indicated that the runway pavements get very hot in the summer. The airport uses pavement sensors to monitor the temperature of the asphalt concrete and indicated that in summer, the temperature of the asphalt concrete at a depth of 12 inches can easily exceed 145°F (maximum temperature recordable by the equipment) by 9:00 am.

The depression and shoving near Taxiway E (Figure B-1) that occurred in 2006 has not increased since the closing of Taxiway E. Aircraft landing on Runway 10L typically used Taxiway E as a

high speed exit to the terminal area. This would have resulted in some fairly high braking and turning forces in the area of the Taxiway. Since closing this taxiway in 2006, aircraft travel further down the runway to Taxiway D where they are moving more slowly.

There are significant areas of staining on the surface of the pavement (Figures B-2 through B-7). The stains do not coincide with the longitudinal paving joint in the permeable friction course but do appear at regular offsets and may be over joints in the underlying asphalt binder course. The brown coloration of the staining suggest that this might be residual asphalt cement from an emulsion mix of asphalt cement and water being pumped out of the pavement. The cores extracted from the pavement to date show significant deterioration of the asphalt concrete binder course. Additional observations are given in the captions for the photographs taken during the site visit, given here as Figures B-1 through B-9.

Asphalt Concrete Sampling

Asphalt concrete cores have been extracted from Runway 10L/28R for laboratory testing. One core was extracted from Taxiways C and G on either side of Runway 10L/28R to represent the pavement condition in areas that are not surfaced with permeable asphalt surface course. Six cores were extracted at various locations along Runway 10L/28R in areas adjacent to cores already extracted that did not show complete deterioration of the asphalt binder course and stripping of the asphalt cement from the aggregate (Figures B-8 and B-9). Additional pavement condition details are provided in the attached photographs.



Figure B-1. Boise Runway 10L/28R looking toward the intersection of Taxiway E. *This shows a patched area and an initial depression and shoving noticed in 2006. Aircraft landing on Runway 10 used this taxiway for relatively high speed exits to the terminal area. This resulted in some fairly high braking forces. No additional distress has been observed.*



Figure B-2. Boise Runway 10L/28R looking east. *The stained area is offset from the paving joint in the permeable friction course. Airport staff indicate that the staining may be pumping from a joint in the underlying base asphalt concrete.*



Figure B-3. Close-up of the staining on the surface of the pavement at Boise, Idaho Airport.



Figure B-4. Close-up of the staining on the surface of the pavement at Boise, Idaho Airport.



Figure B-5. Boise Runway 10L/28R near Taxiway B. *Area to the right has older permeable surface course showing some signs of raveling. Pavement in the left of the photograph was paved in 2000.*



Figure B-6. Boise Runway 10L/28R north of Taxiway M. *Note the staining along the longitudinal joint and low severity cracking in the foreground.*



Figure B-7. Runout area north of Boise Runway 28R near the south edge of the pavement. *Severe staining, cracking and some raveling of the asphalt concrete. The surface in this area is P-401 asphalt concrete.*



Figure B-8. Close-up of an asphalt sample taken from a core of the runway at Boise Airport. *Note the complete stripping of the asphalt cement from the aggregate.*



Figure B-9. Typical core taken from Boise Runway 10L/28R. *The permeable surface course is on the right side of the photograph and is intact. Directly below the surface the asphalt concrete is stripped and deteriorated. The asphalt in the center of the box is the old surface below the asphalt concrete placed in 2000.*

Boston Logan International Airport

The Boston Logan International Airport (BOS) has accommodated scheduled air traffic since 1927. The airport is located on 2,400 acres of property directly east of downtown Boston on primarily reclaimed land in the Boston Harbor. Logan airport is one of the 20 busiest airports in the United States with over 400,000 aircraft operations servicing over 27 million passengers per year. The airport has 6 asphalt concrete surfaced runways ranging in length from 2,557 to 10,083 feet.

Climate

The Boston area averages about 42 inches of precipitation (rain and liquid equivalent of frozen precipitation), including 42 inches of snowfall per year. Temperatures are typical of the east coast with an average summer high of 82°F and average winter low of 22°F. The pavements at Logan airport are subject to significant moisture from rainfall, snowfall and fog given its proximity to the Atlantic Ocean.

Deicing Chemical Use

Aircraft deicing and anti-icing has traditionally taken place at the gates at Logan airport. In 2007, a dedicated deicing facility was constructed near the Runway 14 end of Runway 14/32. This deicing facility is intended to permit additional application of deicing chemicals for aircraft that have a long taxi to Runways 4 and 14.

In the past, the airport has typically used a mix of ethylene glycol and urea for pavement deicing. More recently, potassium acetate has been used.

Pavement Condition and Recent Issues

While many of the runway, taxiway and apron pavements at Logan airport are over 50 years old, the majority of the pavement surfaces are less than 20 years old due to resurfacing and rehabilitation treatments. The overall average pavement condition index of the pavements at the airport is 78 which represents very good condition.

Prior to 2004, highway type asphalt concrete mixes were typically specified using AC-20 asphalt cement and $\frac{3}{4}$ inch top size aggregate. These mixes were prone to rutting and required significant maintenance. In addition, significant stripping of the asphalt concrete was observed in the asphalt concrete mixes placed in the apron areas around the terminal. These mixes contained asphalt cement and a Redicote liquid anti-stripping additive from Bitumar in Montreal, Canada. As a result of the general poor performance of the asphalt concrete, new mixes were designed for the airport which included the use of recycled asphalt concrete, latex modified binders, and 1 percent hydrated lime as an anti-stripping agent.

Logan airport is also very progressive in trying new technologies and has constructed test sections using Citgoflex FR fuel resistant asphalt concrete mixes and warm mix asphalt (Sasobit).

Site Visit Observations

The primary pavements at Logan airport are in good overall condition. Based on the most recent pavement management report, the average pavement condition index (PCI) of the runways is 89 (excellent), taxiways is 80 (very good) and aprons is 73 (very good). The most prevalent distresses observed include low to moderate severity raveling and random cracking. The raveling and cracking is most prevalent in the areas that were overlaid between 2000 and 2002 using the AC-20 from Bitumar with the Redicote liquid anti-stripping additive. The majority of this asphalt concrete was placed on Taxiway K (Figures B-10 through B-12) which is the primary access taxiway around the main apron. Additional specific site visit observations are provided in the captions for the photographs, given in Figure B-10 through B-24.

Asphalt Concrete Sampling

Asphalt concrete cores were extracted from the areas shown in Photographs B-10 through B-15. The cores were taken to be representative of older pavement (1981) which has been exposed to substantial quantities of deicing chemicals and from the asphalt concrete placed in 2000-2001 that has shown poor performance with significant stripping and raveling.



Figure B-10. US Airways Cargo area apron and south outer Taxiway K, Boston Logan Airport. Milled and overlaid with P-401 asphalt concrete in 2000. Note the coarse appears and cracking of the asphalt concrete.



Figure B-11. US Airways Cargo area apron and south outer Taxiway K, Boston Logan Airport. Milled and overlaid with P-401 asphalt concrete in 2000. Note the cracked, segregated and raveled surface.



Figure B-12. FedEx Cargo area apron and south outer Taxiway K, Boston Logan Airport. Milled and overlaid with P-401 asphalt concrete in 2000. Asphalt concrete shows low severity cracking and raveling.



Figure B-13. Apron area in front of Boston Logan B Concourse near Gate B-28. *This pavement was rehabilitated in 1981 consisting of a mill and overlay of 4 inches of P-401 asphalt concrete. Some of the cracking has been sealed but there are also many unsealed cracks. Aircraft anti-icing chemicals run onto this area of asphalt from the adjacent concrete apron pavement.*



Figure B-14. Apron area in front of Boston Logan B Concourse near Gate B-28. *Random pavement surface cracking. Photograph taken in a low area of the pavement. Deicing fluid from the gate deicing operation flows onto the asphalt concrete at this location. White dot indicates core location.*



Figure B-15. Apron area in front of Boston Logan B Concourse near Gate B-28. *Photograph taken in a high area of the pavement. Deicing fluid from the gate dicing operation likely flows onto this area but not as much as the location shown in Photograph 2. White dot indicates core location.*



Figure B-16. Inner Taxiway area in front of Boston Logan B Concourse near Air Canada Gate 3. *This area was milled and overlaid with 3 inches of P-401 asphalt concrete in 2000. Note the significant cracking, raveling and exposed aggregate.*



Figure B-17. Inner Taxiway area in front of Boston Logan B Concourse near Air Canada Gate 3. *Close-up of location shown in Figure B-16.*



Figure B-18. Inner Taxiway area in front of Boston Logan B Concourse near Air Canada Gate 3. *Further close-up of location shown in B-16.*



Figure B-19. Taxiway J and asphalt overrun at the end of Runway 14, Boston Logan Airport. *Asphalt concrete pavement constructed in 2006. No evidence of any raveling or stripping.*



Figure B-20. Boston Logan Runway 06/27 near Taxiway C. *Moderate severity raveling of the transverse grooved asphalt concrete surface.*



Figure B-21. Boston Logan Runway 06/27 near Taxiway C. *Close-up of location shown in Photograph B-20. Note the missing fine aggregate particles and stripping of the asphalt cement from the coarse aggregates.*



Figure B-22. Boston Logan Taxiway K near Taxiway L. Mill and overlay of 2.5 inches of P-401 asphalt concrete in 2001. This asphalt concrete includes the Bitumar AC-20 and liquid antistripping additive. Note the raveling and stripping of the asphalt cement from the coarse aggregate.



Figure B-23. Boston Logan Taxiway K near Taxiway L. Closeup of area shown in Photograph B-22.



Figure B-24. Boston Logan Taxiway K between Taxiway N and Taxiway Z. Microsurfacing was placed over the raveled and stripped asphalt concrete.

Colorado Springs Municipal Airport

The Colorado Springs Municipal Airport (COS) is co-located with Peterson Air Force Base approximately six miles southeast of the central business district of the City of Colorado Springs in Colorado. It is the second busiest airport in Colorado. The airport has three primary runways (two surfaced with asphalt concrete and one with concrete). A new terminal was opened in October of 1994. The elevation of the airport is 6,187 ft above sea level. Figure B-25 is a schematic of the Airport runways and taxiways.

Climate

The Colorado Springs area averages about 15.5 inches precipitation (rain and liquid equivalent of frozen precipitation) including 37 inches of snowfall per year. Temperatures are typically moderate with January lows on the order of 14°F and July highs in the order of 85°F.

Deicing Chemical Use

Aircraft deicing and anti-icing takes place at the gates at COS airport. Pavement anti-icing and deicing is completed using liquid potassium acetate using spray bar applicators. The airport maintenance forces also apply sand for some low volume taxiways to improve surface friction during the winter. Potassium acetate is primarily used for Runway 17L/35R and on Taxiways Foxtrot, Golf and Hotel. Deicing chemicals are not generally used for the asphalt concrete surfaced Runway 12/30. Due to the grade of the Runway 30 end area, chemicals used on Taxiway Foxtrot drain onto the end of the runway. Potassium acetate has been used at the airport since 1999.

Pavement Condition and Recent Issues

The majority of the primary airside pavements at COS have exposed concrete surfaces. Runway 17R/35L left was resurfaced in 2003 and is in excellent condition. A GSB 88 fog seal has been applied to the runway surface. The runway surface has been grooved transversely. The only significant surface distress observed was low severity construction joint cracking. Runway 12/30 was last rehabilitated in 1992 and is beginning to show initial signs of distress. Low to moderate severity construction joint cracking with secondary alligator type cracking and raveling is prevalent. Many of the cracks have been sealed using hot-poured rubberized asphalt sealant using over-banding techniques. The airport operations personnel indicated that the Runway 12/30 asphalt concrete included crumb rubber as a modifier. They were unsure of which layers contained the crumb rubber. The runway surface has been grooved transversely.

Site Visit Observations

There appears to be significant ASR type damage to the concrete pavements. Airport operations personnel indicated that it is very difficult to find aggregate in the Colorado Springs area that is not susceptible to ASR. It is manifested by closely spaced parallel cracking at the concrete joints. The Runway 17R/35L pavement is relatively new and did not show any obvious signs of damage due to deicing chemical use. The cracking, raveling and staining of the asphalt concrete at the Runway 30 end area new Taxiway Foxtrot could be related to the deicing chemicals and an area was selected for coring. Additional specific site visit observations are provided in captions to the photographs, shown in Figures B-26 through B-37.

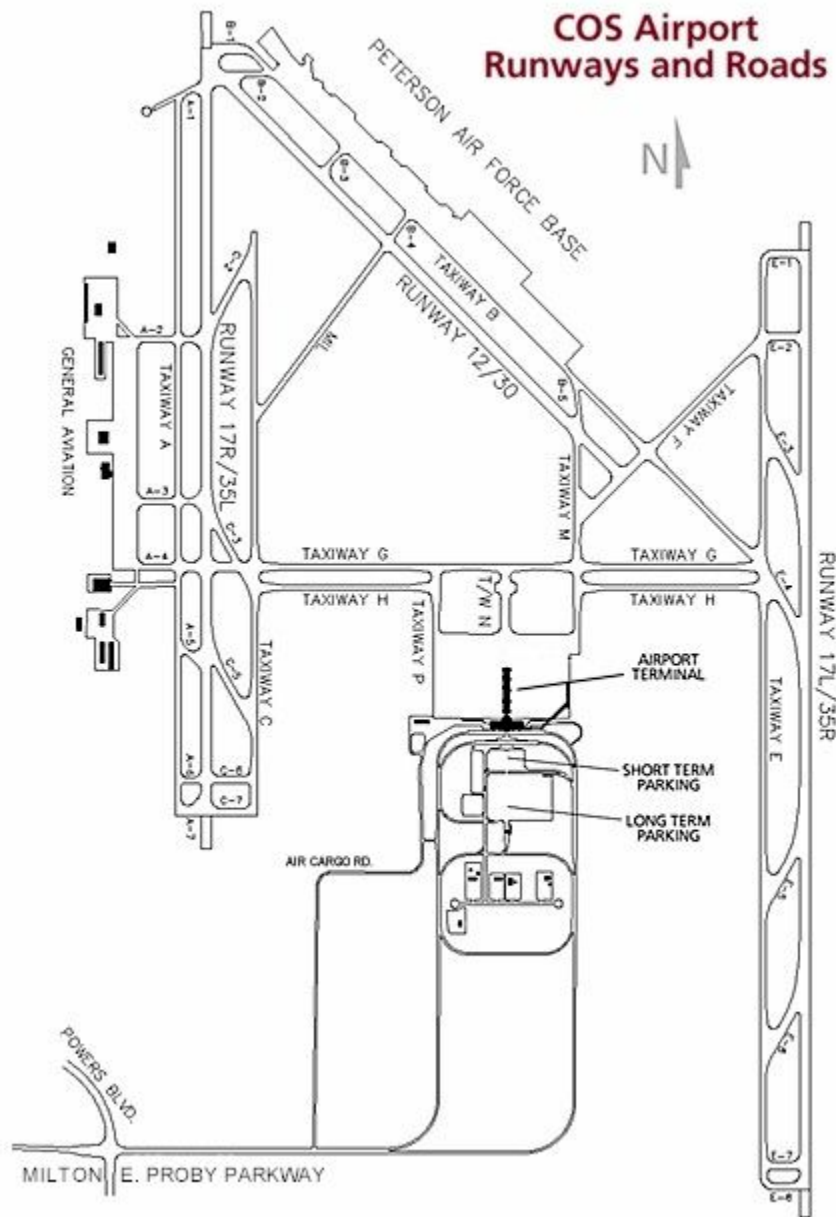


Figure B-25. Schematic of Colorado Springs Municipal Airport.



Figure B-26. Colorado Springs Runway 17L/35R looking north from Taxiway Hotel.



Figure B-27. Colorado Springs Runway 17L/35R looking east at intersection with Taxiway Hotel. *The pavement is new and the only distresses present at this location are low severity longitudinal construction joint cracks.*



Figure B-28. Close-up of low severity longitudinal construction joint crack on Colorado Springs Runway 17L/35R.



Figure B-29. Close-up of slight segregation and secondary cracking near the low severity longitudinal construction joint crack on Runway 17L/35R, Colorado Springs Airport.



Figure B-30. Colorado Springs Runway 12/30 Looking north from the Runway 30 End. Most cracks including the primary longitudinal construction joint cracking has been sealed with hot-poured rubberized asphalt using an over-banding technique.

I



Figure B-31. Large over-band of crack on Colorado Springs Runway 12/30.



Figure B-32. Mid-mat longitudinal cracking and moderate severity transverse crack near the south end of Runway 30, Colorado Springs Airport.



Figure B-33. Close-up of raveling and cracking about 10 feet west of the Runway 30 centerline, Colorado Springs Airport. *Location selected for coring.*



Figure B-34. Cracking, raveling and patching on Runway 30 near Taxiway Foxtrot, Colorado Springs Airport.



Figure B-35. Runway 12/30 looking north at Taxiway MIL, Colorado Springs Airport.



Figure B-36. Runway 12/30 looking south at Taxiway MIL, Colorado Springs Airport. *Note the over-band crack sealing approximately 10 feet west of the runway centerline in the primary wheel path area. Low severity fatigue type cracking on either side of the sealed crack.*

Figure B-37. Close-up of the area shown in Photograph B-36.

Asphalt Concrete Sampling

Asphalt concrete cores were extracted from the pavement in the areas shown in Photographs 40 and 41. Cores were taken near the distressed area and an adjacent area showing no distress. Twelve, six-inch diameter cores were extracted from runway 12/30 (south-north) at Colorado Springs airbase. Cores 1-9 were taken from locations with no distresses and 10-12 were taken from locations with alligator cracking and brown residue deposition:

- Cores 1-4 were taken at approximately 80 ft from the start of the runway (most southern point) and just to the west of the runway's center.
- Cores 5-7 taken at approximately 100 ft and just to the west of the runway's center.
- Cores 8-9 taken at about 125 ft and just to the west of the runway's center.
- Cores 10-12 were taken at about 300 ft from the start and just to the east of the runway where longitudinal alligator cracking were present.

Friedman Memorial Airport

Friedman Memorial Airport, formally know as the Sun Valley Airport (SUN) is a commercial and general aviation airport in the town of Hailey in Blaine County, Idaho. The airport has one runway, 6,952 in length.

Climate

The area surrounding the Friedman Airport averages about 16 inches of precipitation (rain and liquid equivalent of frozen precipitation), which includes 81 inches of snowfall per year.

Temperatures range from an average summer high temperature of 70°F and average winter low temperature of 12°F.

Pavement Condition and Recent Issues

In 2006, pavement deterioration in the form of cracking, raveling and rutting was observed along the south end of the apron in the primary areas trafficked by aircraft approaching the terminal. In addition, some cracking and deterioration of the asphalt concrete was noticed at the north end of the apron. Pavement cores taken in these areas revealed significant stripping and deterioration of the asphalt concrete.

Runway 13/31 and the airport apron pavement were reconstructed in the spring/summer of 2007. The rehabilitation treatment included milling, expanded asphalt stabilization and an overlay.

Site Visit Observations

A site visit to this airport was not scheduled as the construction work had already been completed and there was nothing of the original pavement to see. Photographs of the pavement, as shown in Figures B-38 through B-51, were provided by Toothman-Orton Engineering Co. of Boise, Idaho.

Asphalt Concrete Sampling

The airport arranged to have eight cores extracted from the apron pavement prior to the reconstruction work. Details on the cores are shown in the attached photographs. Although it was intended that some of these cores be subjected to the IT test, none of the cores were in good enough condition to perform this procedure.



Figure B-38. Apron pavement looking north, Friedman Memorial Airport. Alligator cracking at a longitudinal construction joint.



Figure B-39. Apron pavement looking north, Friedman Memorial Airport. Alligator cracking and rutting (transverse to the photograph) on the apron in front of the terminal building.



Figure B-40. Apron pavement looking east, Friedman Memorial Airport. Rutting and depressions in the apron pavement in front of the terminal building.



Figure B-41. Apron pavement looking west towards the terminal building, Friedman Memorial Airport. Rutting due to aircraft gear loading in front of the terminal building.



Figure B-42. Close-up of cracking and raveling of the pavement surface.



Figure B-43. Close-up of cracking and raveling of the asphalt concrete at the cracks.



Figure B-44. Core #1 from Friedman Memorial Airport. *Note deterioration of the asphalt concrete between the surface and binder course lifts.*



Figure B-45. Core #2 from Friedman Memorial Airport. *Note deterioration of the asphalt concrete between the surface and binder course lifts. Note the stripping of the asphalt cement from the aggregate at the surface of the binder course lift.*



Figure B-46. Core #3 from Friedman Memorial Airport. *The core was not able to be extracted in a solid piece. Note the stripping of the asphalt cement from the aggregate.*



Figure B-47. Core #4 from Friedman Memorial Airport. *Surface and binder course has lost some of the aggregate.*



Figure B-48. Core #4 from Friedman Memorial Airport. *Close-up of the bottom of the surface course life (left side of photograph) and top of the binder course life (right side of the photograph). Note the stripping of the asphalt cement from the coarse aggregate.*



Figure B-49. Core #5 from Friedman Memorial Airport. *Intact surface and binder course. Some deterioration between the lifts is evident on the left side of the photograph.*



Figure B-50. Core #6 from Friedman Memorial Airport. *Surface and binder course intact. Some open structure is evident at the bottom of the core but may be due to poor compaction of the lower lift.*



Figure B-51. Core #7 from Friedman Memorial Airport. *Surface and binder course intact. Both the surface and binder course have fairly large aggregate.*

**APPENDIX B:
FOURIER TRANSFORM INFRARED SPECTROSCOPY
MEASUREMENTS**

INTRODUCTION

Differential FTIR functional group analysis can be used to determine the concentration of various chemical functional groups found in asphalt [Petersen, 1986]. Some of these functional groups, like sulfoxide and carbonyl due to ketone formation can be used to help determine the extent of aging in asphalt. The goal of the work described here is to determine whether FTIR functional group analysis can be used to identify a “chemical flag” for deicer induced damage. In particular, samples were pressure aging vessel (PAV) aged in the presence of various deicer chemicals and analyzed. Of particular interest is whether increased carboxylate salt formation due to reaction of deicing chemical and asphalt is a result of deicer induced damage and if FTIR evidence of such can be used to flag a pavement for possible failure. In addition, FTIR spectra were also evaluated for an increase in polycyclic aromatic hydrocarbons (PAH) as has been indicated in other deicing studies [Alatypko 2005; Edwards and Aurstad 2000].

EXPERIMENTAL

FTIR Analysis

All spectroscopic analyses were performed using a PerkinElmer Spectrum One Fourier Transform Infrared Spectrometer. All FTIR spectra of asphalt were obtained using dry tetrahydrofuran (THF). THF was dried by passing over a column of activated basic alumina and storing over molecular sieves.

Materials

Briquette size specimens for moisture damage were prepared from two aggregates and two asphalts received from AAT:

Aggregates

- Virginia diabase, crushed fines
- Virginia sand
- Mississippi chert/gravel, crushed fines
- Mississippi sand

Asphalt

- Citgo PG 58 - 28
- Citgo PG 64 - 22

The minus #20 to plus #35 (-20+35) material for each aggregate received was separated by sieving and then thoroughly washed. The washed material was dried overnight at 150°C and combined as follows:

- 40% Virginia diabase crushed fines, 60% Virginia sand
- 47% Mississippi chert/gravel crushed fines, 53% Mississippi sand

The briquettes were prepared with a binder content of 5% by mass of total mix.

Deicer Chemicals

2% (by mass) aqueous solutions of potassium acetate (CH_3COOK , reagent grade, >99.0%, Sigma-Aldrich), 2% and 35% aqueous solutions of sodium formate (HCOONa , reagent grade, 97%, Sigma-Aldrich), and 2% and 35% aqueous solutions of propylene glycol were prepared as model deicer solutions. Deionized water was used to prepare all aqueous solutions

FTIR – Sample Preparation and Testing

Briquette samples of desired asphalt/aggregate mixtures were compacted following the same procedure as in ultrasonic horn analysis. Some of these briquettes were then immersed in a 2% deicer chemical solution and saturated at room temperature under vacuum (460 mmHg) for one hour. Briquette mixtures were PAV aged in one of three different environments: Dry, saturated with distilled water, or saturated with an aqueous 2% deicer chemical solution. Dry samples were PAV aged in the absence of any water source or vacuum saturated samples. After saturation, the briquettes were PAV aged for 96 hours at 20 atm and 60° C. Following aging, the briquettes were removed from the PAV apparatus, dried in a vacuum oven and individually crushed to aid in extraction of the asphalt. Isolation of the asphalt for analysis was carried out by extracting the briquette material with an 85% toluene/15% ethanol mixture. The solvents were then removed using a rotary evaporator and the resulting asphalt sample was re-dissolved in toluene and transferred to a vial used for weighing samples into the appropriate flasks.

The aged asphalt was analyzed for an increase in carboxylate salts utilizing differential FTIR analysis as previously reported by Petersen and coworkers [Petersen, 1986]. In addition, analysis of an abnormal increase of aromatic C-H stretches was performed in an attempt to identify a flag for increased polycyclic aromatic hydrocarbons.

PHASE I RESULTS

In phase 1, initial evaluation of the results from functional group analysis indicated that this method would be a useful tool for using carboxylate salts as a chemical flag for deicer induced damage. The phase 1 analysis included briquette samples that were PAV aged in a dry environment after vacuum saturating with distilled water, vacuum saturating with a 2% potassium acetate (KOAc) solution, or vacuum saturating with a 2% sodium formate (NaOf) solution. As expected, the data indicate that essentially no carboxylate salt formation occurred when samples were PAV aged in a dry environment or in the presence of deionized water with the exception of one sample, AC_1189/VA. Most of the briquette samples aged in the presence of the deicing salts exhibited an increase in carboxylate salt concentration as observed by FTIR. As was the case with the samples aged in the absence of salt, however, there was one unlikely exception, AC_1364/VA in NaOf, which showed no evidence of age-induced carboxylate salt formation. As a result, all of the NaOf sample analyses were to be repeated during the phase 2 work. It is also important to point out that the analyses were performed in the presence of only one concentration of deicer chemical; not enough to form a complete conclusion regarding a trend in carboxylate salt formation. Interestingly, our results indicated an increased presence of carboxylate salts in the FTIR spectra for samples aged with KOAc relative to NaOf. This is counter to the observation from previous research that NaOf appears to have a greater affect on an asphalt-aggregate mixture. This observation, in addition to the exceptions that were observed

in the carboxylate salt concentrations pushed the phase 2 work in terms of determining whether the results obtained in phase 1 were indeed demonstrating carboxylate salt formation in the asphalt or were simply due to presence of dissolved deicer chemicals in the recovered asphalt.

Another aspect of the phase 1 FTIR work was the analysis of all spectra for any abnormal characteristics that could be attributed to an increased level of polycyclic aromatic hydrocarbons (PAHs). Unfortunately this work was inconclusive. Due to the presence of aromatics naturally found in most asphalts, as well as the peaks attributed to tetrahydrofuran, the identification of PAHs was not possible within any appreciable level of confidence. It could be useful to attempt to identify PAHs in airfield asphalt pavements using other analytical tools like, for example, gas chromatography or attenuated total reflectance FTIR.

At the conclusion of the phase 1 work further work was proposed for phase 2. A need was indicated for additional analyses to validate the method using a wider concentration range of deicing solutions as well as additional deicing chemicals. Since all of the mixes used for this project at this point appeared to be prone to stripping, other asphalt aggregate mixes would be needed; a marginal stripping mix as well as one known to resist stripping should be studied in particular.

PHASE II RESULTS

The phase 2 FTIR functional group analysis included some repeat analyses of phase 1 materials as well as analysis of additional, aqueous deicer chemical solutions with the same asphalt aggregate mixtures. The additional deicer solutions were 35% NaOf, 35% KOAc, and both 2% and 35% propylene glycol solutions. Briquette samples were prepared and aged identically to phase 1, as were the asphalt FTIR samples. The results of the phase 2 FTIR analyses are shown in Figures B-1 through B-4. Each figure is a plot of peak area at 1730 cm^{-1} , which is proportional to carboxylate salt concentration, versus the method by which a sample was aged/treated. Keep in mind that no samples were treated and aged in the presence of a 35% KOAc solution.

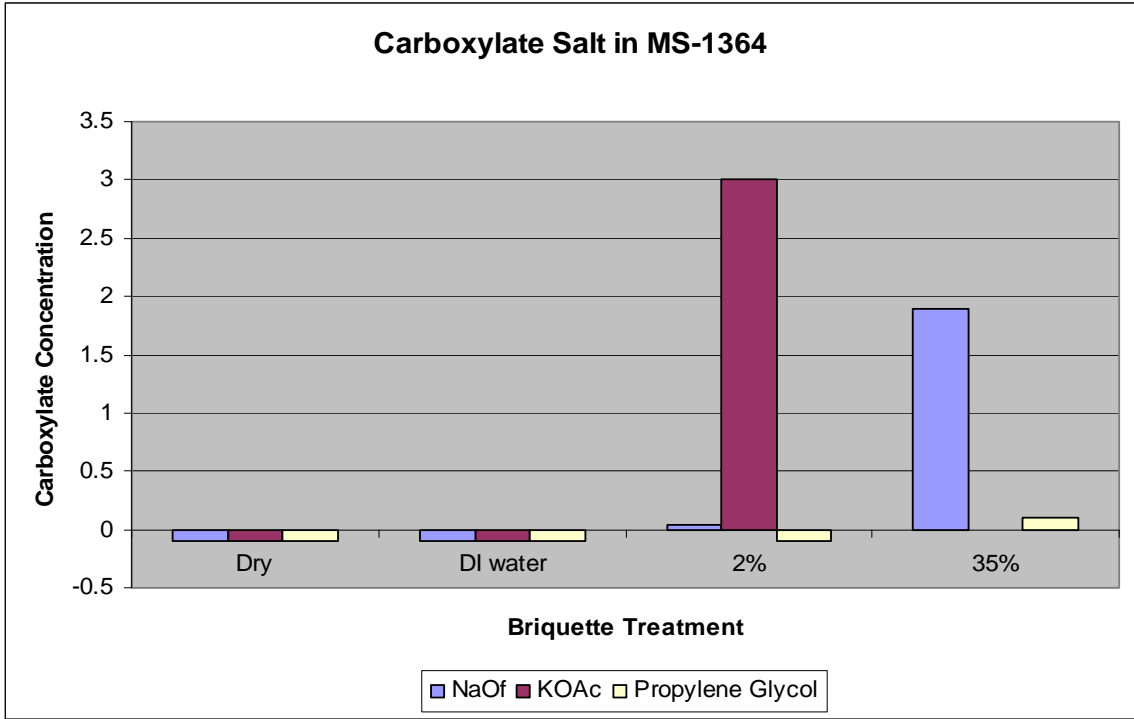


Figure B-1. Carboxylate salt concentration found in sample MS-1364.

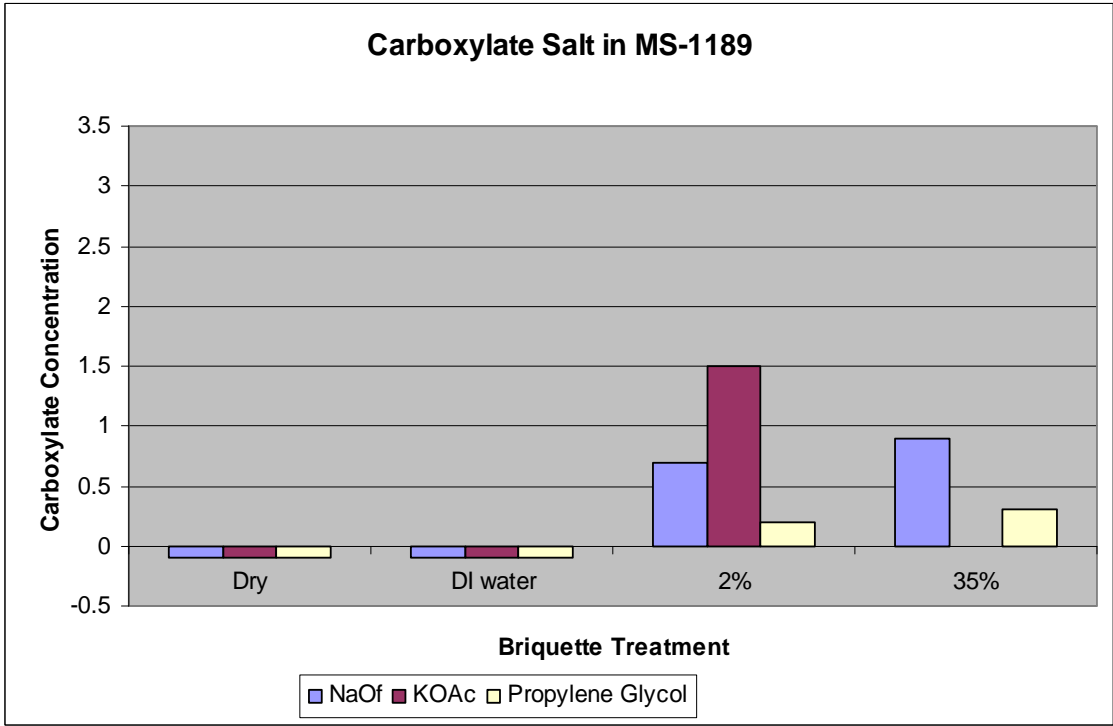


Figure B-2. Carboxylate salt concentration found in sample MS-1189.

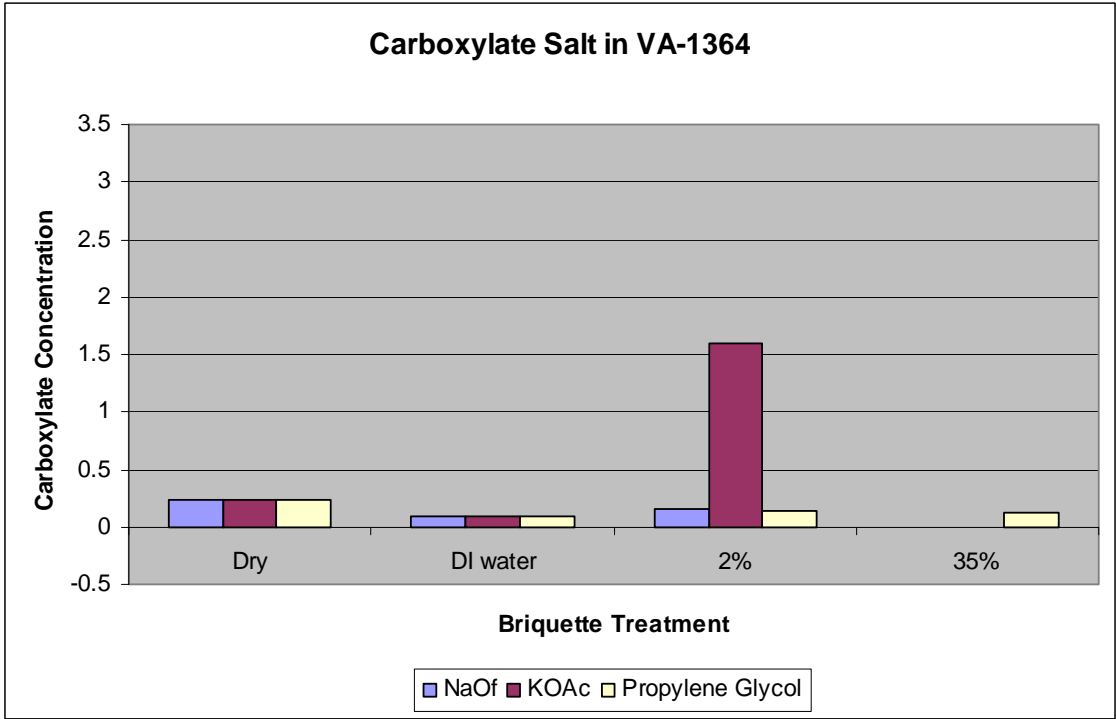


Figure B-3. Carboxylate salt concentration found in sample VA-1364.

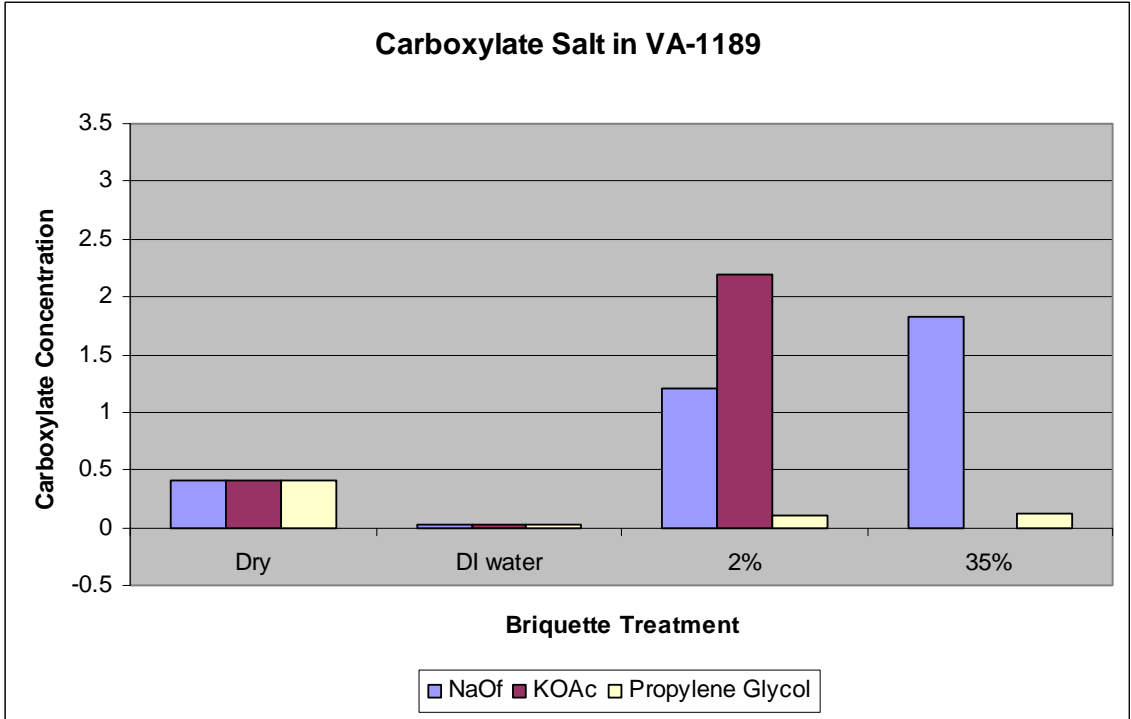


Figure B-4. Carboxylate salt concentration found in sample VA-1189.

The first thing to point out from Figures B-1 through B-4 is the lack of any clear trend in carboxylate salt concentration based on the deicer chemical used. Regardless of deicer chemical, asphalt, or aggregate source the salt concentration of the extracted samples varied greatly. The exception is propylene glycol. As expected, the presence of propylene glycol in the aging process did not result in excess carboxylate salt formation. Propylene glycol is a fairly simple organic molecule lacking the presence of metal anions like sodium or potassium that could react with the carboxylic acids present in the asphalt. Carboxylate salt concentration for dry aged samples and those aged in the presence of DI water was, essentially, zero. Judging by the numbers observed for these measurements, the area under the IR absorbance at 1730 cm^{-1} ranged from -0.1 to 0.4. This indicates, for this use of the functional group analysis method, that a change in area of 0.5 or greater indicates a significant difference in salt concentration.

The data obtained for the samples aged in 2% NaOf and KOAc solutions in phase 1 indicated that a greater amount of KOAc was present than NaOf after sample extraction. Unfortunately, one of the samples aged in NaOf indicated that no carboxylate salt was present after extraction. This was somewhat troubling, but the work was revisited in phase 2. The results in Figures B-1 through B-4 are representative of what was found from reproducing the work with the 2% deicer chemical solutions. The amount of salts presents in samples aged in the presence of KOAc was still greater in all four cases. An increase in carboxylate salt formation was observed in the samples aged in 35 % solutions of NaOf relative to the 2% solutions in three of the four samples. Interestingly, three of the samples aged in the presence of 35% NaOf solutions had less carboxylate salt present than the 2% KOAc solutions. This was a troubling observation when coupled with the absence of carboxylate salt for the VA-1364 sample aged in the presence of 35% NaOf. All samples were aged in duplicate so that, in the case of an anomaly, a second sample could be extracted and subjected to functional group analysis. The duplicate VA-1364 samples aged in NaOf were also analyzed and the results are shown in Figure b-5 below.

The figure shows a problem inherent in this study. In the second run, the sample aged in a 2% NaOf solution contained more carboxylate salt than the 35% sample. This indicates an issue in the processing of the samples prior to subjecting them to FTIR analysis. There is no clear explanation for the differences between the first run and the second run results. Different scientists performed the extractions, though using the same procedure. The first run results were anomalous in relation to other analyses performed in parallel; therefore, the second run should be recognized as typical for these experiments. Basically, the carboxylate salts observed in the functional group analysis are most likely the result of the presence of deicing chemicals in the extracted sample rather than products of a reaction between deicing chemicals and the asphalt.

In the sample work-up, a crushed briquette is extracted utilizing an 85%/15% toluene/ethanol mixture followed by solvent removal and redissolution in toluene to transfer to IR preparatory flasks where the toluene is removed and the sample is redissolved in THF. Initial studies of the 2% samples did not indicate an appreciable amount of deicing chemical present. The salts were not visibly observed upon extraction, after dissolution in toluene, or in the THF-dissolved IR sample. The presence of a marked amount of salt in THF would be visible as it settled to the bottom of the flask or in the IR spectrum. IR of both NaOf and KOAc in THF did not indicate any appreciable solubility of the salts either. However, both of these salts are expected to be slightly soluble in ethanol so their presence is not totally surprising. In fact, according to the

CRC [2008], KOAc is more soluble than NaOf by an order of magnitude which explains the observations in Figures B-1 through B-4 above. A simple filtration of the sample following extraction and redissolving in toluene did not have any observable effects on the outcome of these experiments. Even though the deicing salts were not soluble in THF they would still react with the silylating reagent used for the determination of carboxylate salts in this method. These products are THF soluble and would be evident in the IR spectrum. Centrifuging dissolved samples is one way that might remove excess salt, though any soluble deicer chemical would remain. An aqueous-organic extraction of the salt impurities from the asphalt sample would be the most effective way to remove the remaining salts. This is a somewhat difficult, time consuming process that was not within the scope of this research. It would also not be recommended for use in a standard test due to the likelihood of compromising the asphalt sample because of asphalt emulsification and technician error.

The use of functional group analysis could still prove to be an asset as a flag for deicer-induced damage, especially of a deteriorated pavement under an overlay. The results above have shown that the use of this method provides an indicator for the presence of excess deicing chemicals when carboxylate salts are the culprit. Unfortunately, it cannot be considered sensitive to change in salt concentrations. If a failed airfield pavement is cored, the sub-surface material could be analyzed for carboxylate salts to demonstrate their presence.

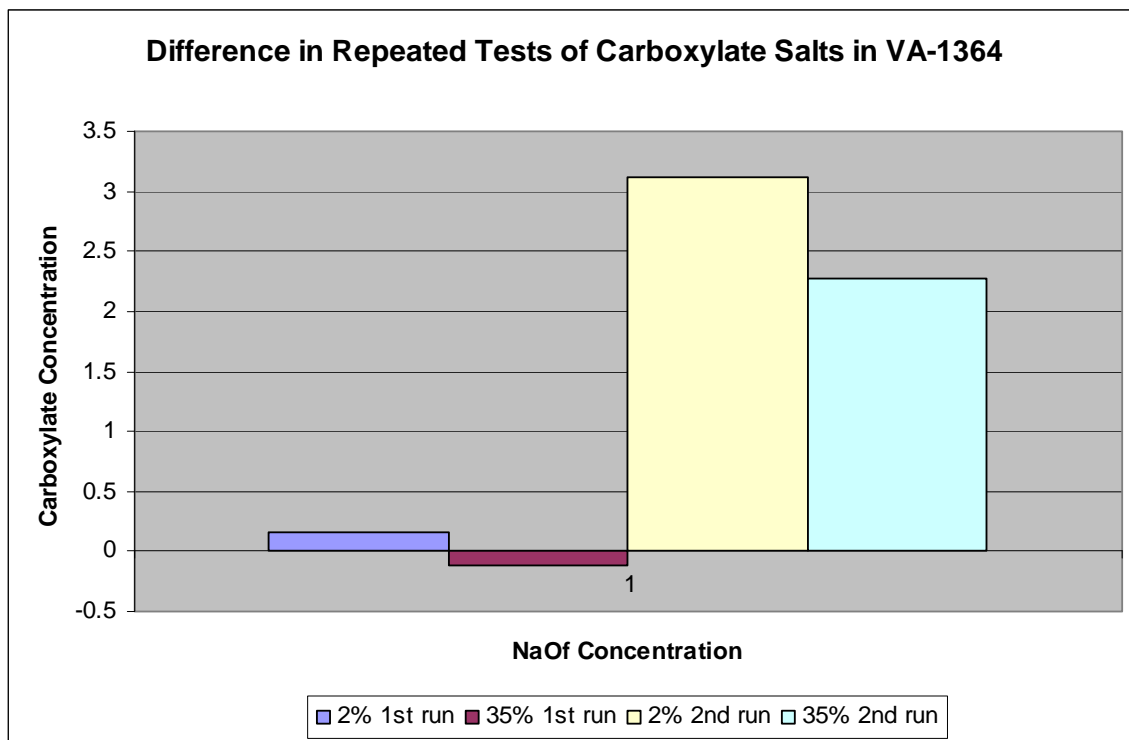


Figure B-5. Carboxylate salt concentration found from repeat analysis of VA-1364.

CONCLUSIONS

FTIR functional group analysis was used to measure carboxylate salt concentration of four different asphalt-aggregate mixes when aged in briquette form in the presence and absence of deicing chemicals. No carboxylate salt formation was observed for 2% and 35% aqueous propylene glycol saturated briquettes following PAV. Extensive amounts of carboxylate salts were observed in various briquettes that were PAV aged in the presence of both aqueous sodium formate and potassium acetate solutions. The observed salts were not formed by reaction of the deicing chemicals with the asphalt, but rather due to the solubility and presence of the deicing chemicals in the FTIR samples. There is no simple, cost effective way to remove these salts from the asphalt samples for analysis to determine whether increased carboxylate salt formation could be used as a flag for deicer induced pavement damage. However, this method could be used in its current form as a routine analysis to determine the presence of deicing salts like sodium formate and potassium acetate in an airfield pavement. FTIR data for each sample was also analyzed to determine whether this method would be useful for indicating increased PAH concentration as a flag for deicer induced damage. Unfortunately, sample and solvent interference in the aromatic region of the IR spectrum make it difficult to obtain any useful information about PAH formation.

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APPENDIX C: SURFACE TENSION MEASUREMENTS AND RELATED ANALYSES

INTRODUCTION

Nuclear magnetic resonance (NMR) imaging methods can be used to obtain asphalt-water interfacial parameters, such as contact angles and surface tensions. These data can then be used to develop a better understanding of asphalt/water interactions and an understanding of what parameters are important for determining the moisture sensitivity of asphalts. The results of this study indicate that deicing chemicals have an affect on the interfacial properties of asphalt and water. In addition, the added affect of increased density of some concentrated aqueous DIAIC solutions could have a marked affect on the penetration of water into asphalt.

MATERIALS

Asphalt samples used were the PG 58-28, and the PG 64-22. These were the same asphalts that were tested in Phase one. Solutions representing deicer salt solutions were made in amber bottles at concentrations of 2 mass percent and 35 mass percent salt in distilled water. The bottles as well as the potassium acetate (KOAc) and sodium formate (NaOf) salts were dried in the oven at 120°C over night. Deicing chemicals were weighed into the dried bottles at a ratio of 1.20 g per 58.80g of distilled water for the 2% by weight solutions and 35.00g per 65.00g of water for the 35% by weight solutions. Solutions were cooled to room temperature before being used. The deicer solutions prepared for NMR imaging measurements are listed in Table C-1. The density and DuNuoy surface tension of each deicer solution (DuNuoy ring tensiometer) are also included in the table. The density of each solution was determined by weight comparison of the deicer solution versus an equal volume of distilled water.

Table C-1. Deicer samples used in the NMR imaging measurements.

Code	Concentration	DuNuoy Surface	Density g/ml
		Tension Dynes/cm	
Water	Water	72.4	1.000
2% SF	2% sodium formate	69.1	1.0144
2% PA	2% potassium acetate	71.3	1.0084
2% PG	2% propylene glycol	68.0	1.0096
35% PG	35 % propylene glycol	51.5	1.0359
35% SF	35% sodium formate	52.6	1.2456
35% PA	35% potassium acetate	65.3	1.1945

SAMPLE PREPARATION

Samples for NMR imaging experiments were prepared by annealing the asphalt at 120°C for one hour, pouring the sample into 23 mm OD glass vials to a depth of 25 mm, and allowing the samples to cool to ambient temperature overnight under a nitrogen atmosphere. After cooling, a 50 µl drop of distilled water was placed on the asphalt surface and a polyethylene cap was placed on the sample vial to minimize headspace above the water drop and prevent evaporation. A photograph of a drop on an asphalt surface is shown in Figure C-1.



Figure C-1. Photograph of a water drop on asphalt surface.

NMR MEASUREMENTS

NMR imaging experiments were carried out at a nominal proton resonance frequency of 200 MHz using a Varian/Chemagnetics microimaging probe. Images were acquired using the spin echo method with an echo time of 10 ms, a pulse delay of 1.0 s, a free induction decay size of 256 data points, 128 phase encodes, 8 acquisitions and a gradient strength of 35 G/cm. Typically, 2 mm thick, single slice images were obtained in a field of view (FOV) of 15 mm. The acquisition time for these experiments was ~ 23 min. The NMR probe temperature was 22°C for these experiments. The different configurations for acquiring an NMR image are shown in Figure C-2. For this project, single slice images were acquired in the YZ plane.

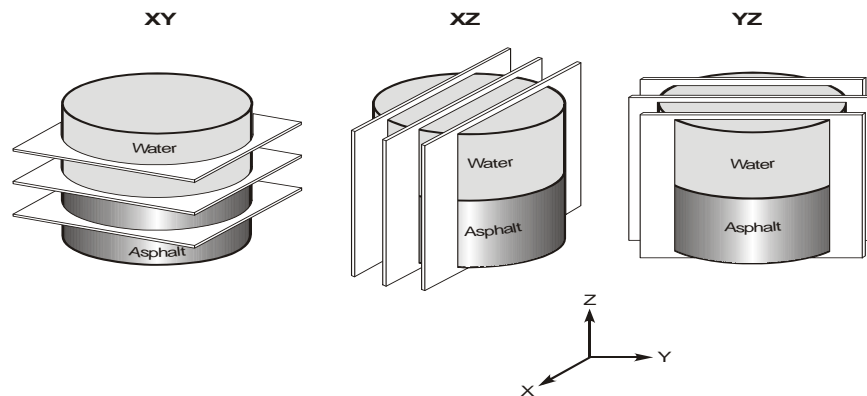


Figure C-2. Illustration of slice geometry used in magnetic resonance imaging.

CONTACT ANGLE MEASUREMENTS FOR ASPHALT-BINDERS AND DEICER SOLUTIONS

Images of each water drop on asphalt were scanned, then transferred to a laboratory computer, and enhanced using Image-Pro Plus 6.2 (2007 Media Cybernetics). Enhancement consisted of increasing the contrast and highlighting the edges of the drop. An example of enhanced images is shown in Figure C-3 for the PG 58-28. The contact angles for the asphalt-water and asphalt-deicer solution interfaces were then measured using the Drop Snake, drop-shape analysis plug-in for ImageJ software from the National Institutes of Health, USA. Drop images were opened in ImageJ and the view was zoomed to 200%. Contact angles measured on the left and right sides of the water drops are given in Table C-2.

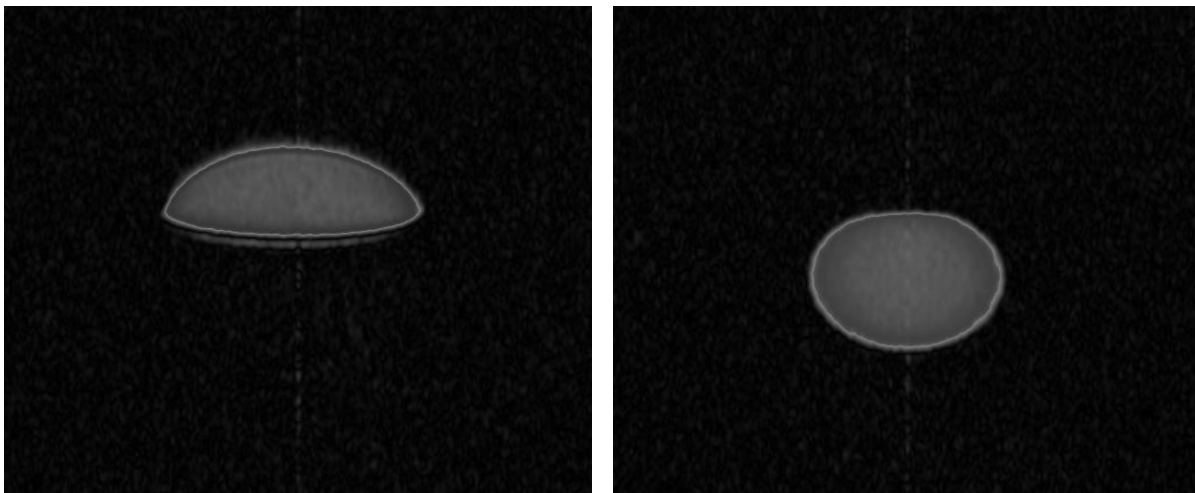


Figure C-3. NMR images of sample 35 % potassium acetate solution in PG 58-28 asphalt binder at 0h and 22h illustrating contrast enhancement.

CALCULATION OF SURFACE TENSION FOR ASPHALT BINDER AND DEICER SOLUTIONS

NMR images of a water drop on an asphalt surface at different stages of descent into an asphalt sample are shown in Figure C-4 along with schematic diagrams that illustrate the corresponding surface tension. Although asphalt is composed in-part by a large number of hydrogen atoms, most are in rigid environments. As such, the NMR line widths for these hydrogen atoms are too broad to provide any image resolution. Therefore, only the mobile protons of the water molecules are evident in the images.

Young's equation is used to describe the surface tension components for a drop of water on an asphalt surface (Figure C-4a) as follows:

$$\gamma_a = \gamma_w \cos\beta(t_1) + \gamma_{aw} \quad (C-1)$$

where γ_{aw} is the asphalt-water interfacial surface tension, γ_w the air-water surface tension, and γ_a the air-asphalt surface tension.

Table C-2. Contact angles measured at the left and right sides of the water drop images.^a

Solution	Left contact angles				Right contact angles			
	$\beta(t_1)$	$\alpha(t_1)$	$\beta(t_2)$	$\alpha(t_2)$	$\beta(t_1)$	$\alpha(t_1)$	$\beta(t_2)$	$\alpha(t_2)$
<i>PG 58-28 Binder</i>								
Water	70	33	0	146	67	49	0	150
2% SF	73	39	0	150	72	46	0	144
2% PA	63	35	0	144	57	49	0	142
2% PG	68	36	0	152	69	45	0	146
35% PG	52	31	0	158	53	30	0	146
35% SF	46	25	0	151	45	29	0	149
35 % PA	68	42	0	132	63	44	0	135
<i>PG 64-22 Binder</i>								
Water	76	16	0	137	83	21	0	155
2% SF	71	22	0	161	72	39	0	155
2% PA	74	20	0	165	73	33	0	147
2% PG	77	18	0	143	73	35	0	149
35% PG	62	22	0	150	73	27	0	134
35% SF	68	21	5	136	78	25	5	142
35 % PA	84	22	9	153	87	34	9	141

^a β refers to the angle above the asphalt surface; α refers to the angle below the asphalt surface, t_1 and t_2 refer to times at which the images were recorded

For the case where the drop of water has sunk below the asphalt surface (Figure 4c) the contact angle would be given by:

$$\gamma_a = \gamma_w + \gamma_{aw} \cos \alpha(t_2) \quad (C-2)$$

In Equations C-1 and C-2, and in subsequent equations, t_1 and t_2 refer to the times at which the initial and final drop images are acquired, respectively. Combining Equations C-1 and C-2 yields a simple expression that can be used to directly calculate the asphalt-water interfacial surface tensions for situations represented by Figures C-4a and C-4c:

$$\gamma_{aw} = \gamma_w \left(\frac{1 - \cos \beta(t_1)}{1 - \cos \alpha(t_2)} \right) \quad (C-3)$$

In going from the situation depicted by the schematic in Fig. C-4a, to the situation depicted by the schematic in Figure C-4c, the drop of water can be described as a dynamic liquid lens, which at any time can be defined by the three contact angles α , β , and θ (Figure C-4b) [Adamson and Gast 1997]. These three contact angles are related to each other and to the asphalt-water

interfacial surface tension, γ_{aw} , the air-water surface tension, γ_w , and the air-asphalt surface tension, γ_a , respectively, by:

$$\gamma_a \cos \theta = \gamma_w \cos \beta + \gamma_{aw} \cos \alpha, \quad (C-4)$$

Equation C-4 is referred to as the Newman triangle [Adamson and Gast 1997].

For liquids of similar densities (as is the case with water and asphalt) the angle θ is small so to a first approximation, $\theta = 0$. By measuring the contact angles of the dynamic lens at two different times, two simultaneous equations are obtained that can be solved simultaneously to give,

$$\gamma_{aw} = \gamma_w \left(\frac{\cos \beta(t_1) - \cos \beta(t_2)}{\cos \alpha(t_2) - \cos \alpha(t_1)} \right) \quad (5)$$

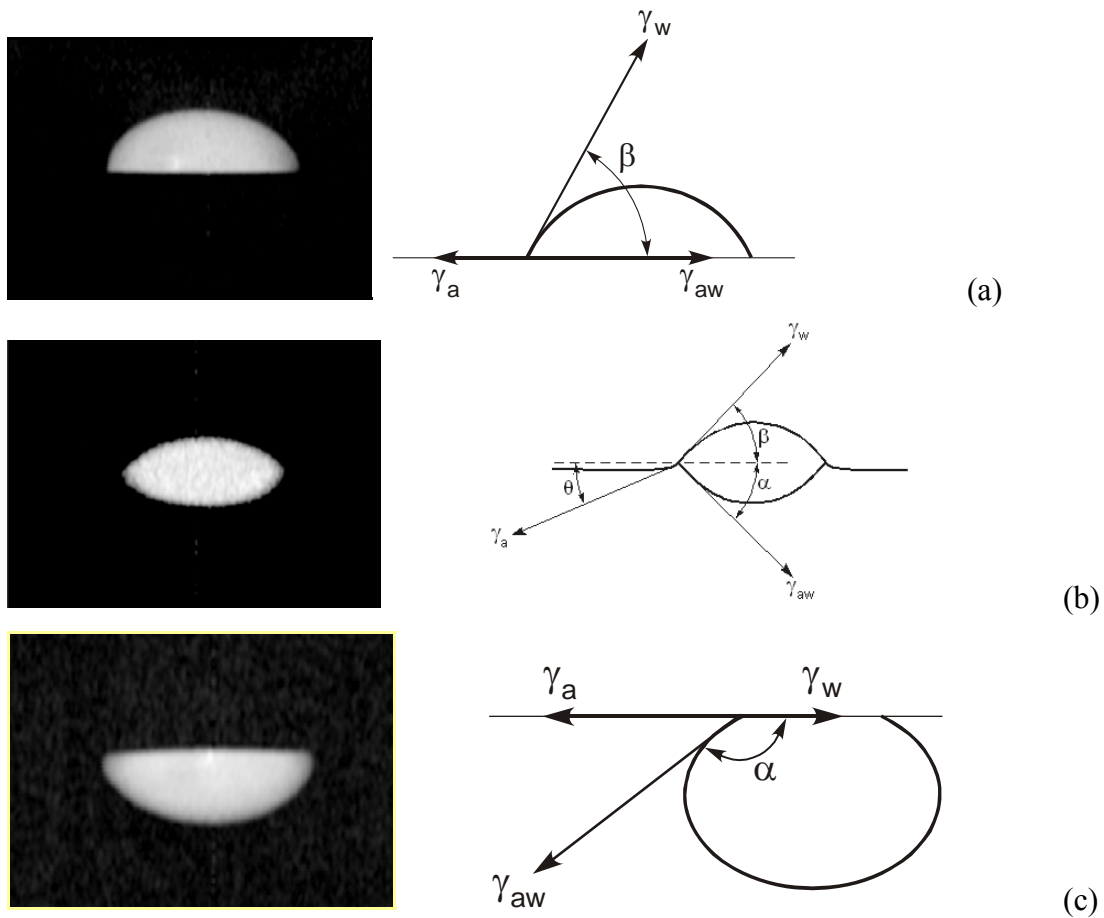


Figure C-4. NMR images of water drops at various stages of sinking and schematic diagrams illustrating surface tension forces.

After measuring the surface tensions of the solution used for the drop by some other method (DuNuoy balance, Wilhelmy plate, etc.) and measuring the contact angles of the droplets on the asphalt by NMR imaging, the asphalt-droplet surface tension can be calculated using Equations C-3 or C-5, depending upon which droplet (b or c) was used for the second measurement. The asphalt-air surface tension can then be calculated from Equations C-1 or C-4. The best situation is to measure the contact angles for droplets a and c because the contributions from the small angles below or above the surface are negligible. It is worth noting that any small angle that is measured for the droplet below the surface (a) or above the surface (c) has little effect on the surface tension values. Indeed, for an angle of 0° and an angle of up to 10° , the value of the cosine only changes by 0.0152 (1.52%).

A period of about 24 h was sufficient time for the droplet to penetrate below the asphalt surface so that good contact angle measurements could be made. The values of the surface tensions calculated from the contact angle measurements are given in Table 3. Surface tensions were calculated from contact angles measured on the left and right sides of the images, as well as from the average value of the asphalt-air surface tension. For several samples there were considerable differences in the left and right contact angles (the PG 58-28 with water, 2 % potassium acetate and 35 % potassium acetate; and the PG 64-22 with water, 2 % propylene glycol and 35 % sodium formate). On the other hand, agreement among the calculated surface tensions was quite good, especially for the PG 58-28. We attribute this to the fact that the surface tensions are determined using two contact angles at the point of contact, and even if the left and right angles are different, the fact that the surface tensions are calculated using two angles partially compensates for these differences.

Previous NMR imaging measurements of the SHRP core asphalts gave values in the range of 26 to 38 dynes/cm for the asphalt-water surface tension and 38 to 48 dynes/cm for the asphalt-air surface tensions (Miknis et al. 2005a, b). The asphalt-water surface tensions of the deicer liquids are somewhat less than those of the asphalt-water surface tensions determined for the SHRP asphalts (Miknis et al. 2005a,b), but still of the same magnitude. The asphalt-water surface tension values for the 35 % propylene glycol and 35 % sodium formate solutions for both asphalt binders (particularly the PG 58-28) are lower than the others in the sample sets

A somewhat disappointing result is the variation of the asphalt-air surface tension values. These values should be the same for all the samples, because the asphalt-air surface tensions are independent of any liquids placed on their surface. It could be that the variations show the reproducibility of the NMR imaging method. A number of things can contribute to this. One is sample preparation to ensure the surface is the same for all samples before adding the water droplet. Also, centering the drop in the sample vial could be a problem. The standard deviations ranged from 4-5 dynes/cm for each asphalt data set. This gives an indication of the reproducibility of the method and what values represent a significant difference in surface tension for a given series.

Table C-3. Surface tensions calculated from NMR contact angle measurements.^{a,b}

Solution	Left Contact Angle		Right Contact Angle		Average	
	Asphalt-H ₂ O ^c	Asphalt-air ^c	Asphalt-H ₂ O ^c	Asphalt-air ^c	Asphalt-H ₂ O ^c	Asphalt-air ^c
<i>PG 58-28 Binder</i>						
Water	29 (25)	49	29 (27)	47	29 (25)	48
2% SF	30 (33)	43	32 (35)	43	31 (33)	43
2% PA	22 (17)	52	20 (11)	54	21 (17)	53
2% PG	26 (25)	46	29 (31)	44	27 (25)	45
35% PG	11 (17)	41	12 (17)	41	12 (17)	41
35% SF	9 (10)	45	9 (10)	45	9 (10)	45
35 % PA	30 (29)	46	26 (23)	48	28 (29)	47
	Average	46		46		46
	Std. dev	4		4		4
<i>PG 64-22 Binder</i>						
Water	33 (25)	49	35 (33)	41	34 (24)	45
2% SF	25 (21)	46	29 (24)	43	27 (20)	44
2% PA	24 (24)	45	27 (23)	46	26 (23)	46
2% PG	31 (28)	44	29 (25)	43	30 (27)	44
35% PG	15 (19)	38	23 (28)	36	19 (18)	37
35% SF	20 (24)	38	25 (22)	33	23 (23)	36
35 % PA	33 (38)	36	39 (44)	35	36 (37)	36
	Average	42		40		41
	Std. dev	5		5		5

^a Drop Snake, left images, 5/21/08; ^b numbers in () are calculated using the average asphalt-air surface energy, ^c all data in Dynes/cm.

The NMR images of the PG 58-28 binder and 35 % sodium formate showed an interesting behavior in that the initial drop has spread considerably (wet) over the asphalt, but after about 22 hrs the drop has contracted and sunk below the asphalt surface. We attribute this behavior to the asphalt, because of its lower surface tension than the water drop, attempting to wet the water drop and therefore pulling the drop into the asphalt (Miknis et al. 2005a). The densities of the 35 % sodium formate and 35 % potassium solutions are significantly greater than pure water so gravity may also play a role. Similar behavior is noted for the PG 58-28/ 35 % propylene glycol system, the PG 64-22/35 % propylene glycol system and the PG 64-22/35 % sodium formate system, but not to as great an extent.

In one set of experiments, images were acquired on the PG 58-28/35 % sodium formate system for up to a period of 497 hrs (~ 20 days). The purpose of these experiments was to observe what happens to the water drop as it falls through the asphalt. These images are shown in Figure C-5. In the lower set of three images, the field of view was changed from 15 mm to 25 mm so that the drop would remain in the field of view as it fell through the asphalt. The last image shows a flattened drop as it rests on the bottom of the sample vial. The fact that the contact angle is greater than 90° indicates that the drop did not displace the asphalt and wet the glass vial.

Instead, there must be a film of asphalt of undetermined thickness between the drop and the glass surface.

As the drop falls into the asphalt, the shape becomes more spherical because the surface-to-volume ratio of the drop is minimized in the hydrophobic medium of the asphalt and because gravity is pulling the drop into the asphalt. Another important observation is that the images show that the water drop retains its shape and does not diffuse into the asphalt in all directions. This is also observed when the drop is above the asphalt surface, and when the drop is partially below the surface. At the molecular level there could be interaction at the water/asphalt interface, but this cannot be resolved with the spin echo imaging sequence employed here. However, the interactions cannot be large since the water drop maintains its shape even after prolonged settling, as in the 497 h image.

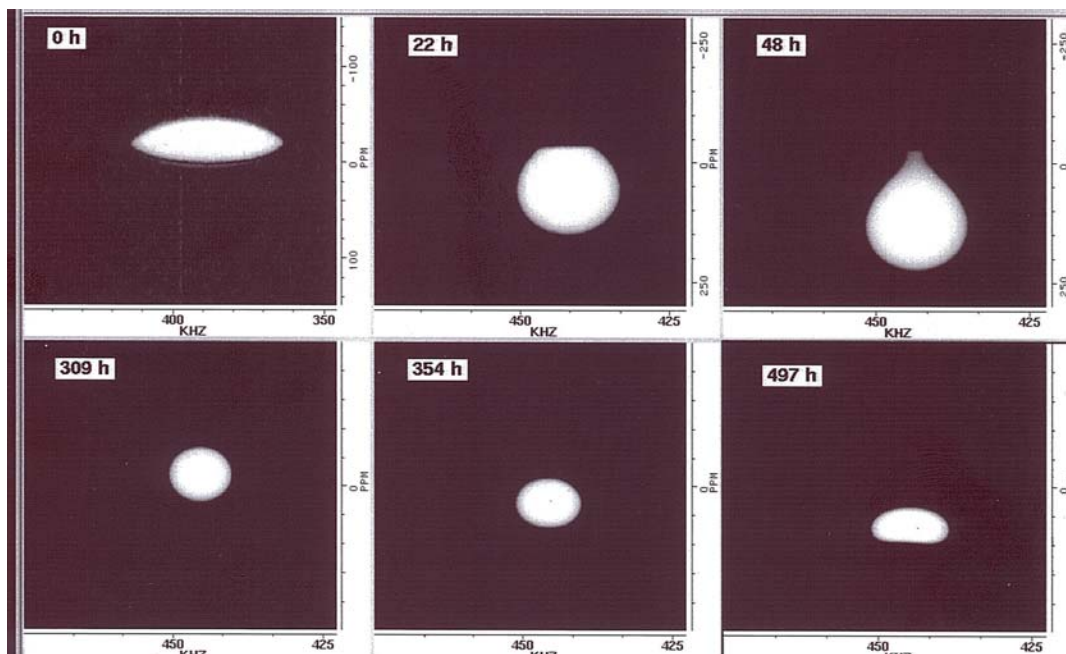


Figure C-5. YZ planar NMR images of a drop of 35 % potassium acetate solution falling through the PG 58-28 asphalt binder.

OTHER INTERFACIAL PARAMETERS FOR ASPHALT BINDER AND DEICER SOLUTIONS

If the asphalt-air and asphalt-water surface tensions are known, additional interfacial parameters can be calculated that might be of use in modeling asphalt-water interactions. Two such parameters are the spreading coefficient and the work of adhesion. The spreading coefficient, S , is a measure of the spontaneity for one liquid to spread on a second liquid or solid surface. More positive values in S denote better wetting systems. The spreading coefficient for water on asphalt is given by,

$$S_{wa} = \gamma_a - \gamma_w - \gamma_{aw} \quad (C-6)$$

and the spreading coefficient for asphalt on water is given by,

$$S_{aw} = \gamma_w - \gamma_a - \gamma_{aw} \quad (C-7)$$

where γ_{aw} is the asphalt-water interfacial surface tension, γ_w is the water-air surface tension, and γ_a is the asphalt-air surface tension. Spreading coefficients for the deicer solutions and asphalt, calculated from the left side images, are given in Table C-4. Note that the spreading coefficients are not simply the negatives of each other.

Table C-4. Interfacial Parameters derived from Surface Tension data.¹

Solution	Spreading Coefficient water on asphalt, S_{wa}^b	Spreading Coefficient asphalt on water, S_{aw}^c	Work of Adhesion, W^a	Hydrophobicity^d
<i>PG 58-28 Binder</i>				
Water	-52	-5	93	0.395
2% SF	-56	-4	83	0.400
2% PA	-43	-5	99	0.225
2% PG	-47	-3	89	0.303
35% PG	-21	-1	82	0.250
35% SF	-17	-1	88	0.164
35 % PA	-48	-10	82	0.361
<i>PG 64-22 Binder</i>				
Water	-56	-9	89	0.368
2% SF	-48	-1	90	0.368
2% PA	-53	-1	89	0.342
2% PG	-54	-6	82	0.368
35% PG	-28	-2	75	0.293
35% SF	-34	-5	71	0.325
35 % PA	-61	-3	70	0.400

^a $W = \gamma_w + \gamma_a - \gamma_{aw}$; ^b $S_{wa} = \gamma_a - \gamma_w - \gamma_{aw}$; ^c $S_{aw} = \gamma_w - \gamma_a - \gamma_{aw}$; ^d drop height/drop diameter

¹ Drop Snake, left images

The work of adhesion is a measure of the work or energy that must be overcome in order to form two surfaces of surface area, A, from an interface.

$$W_{aw} = \gamma_w + \gamma_a - \gamma_{aw} \quad (C-8)$$

where the γ_w , γ_a , and γ_{aw} are the same as in Equations C-6 and C-7. The work of adhesion values are listed in Table 4.

Another parameter is the hydrophobicity, defined as the ratio of the height of the drop to its diameter. Hydrophobicity values are also given in Table C-4. The hydrophobicity and spreading coefficient data for the 35 % propylene glycol and 35 % sodium formate solutions with both asphalt binders reflect the behavior noted in the drop images, i.e., initial spreading of the water drop on the asphalt surface.

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**APPENDIX D: STANDARD TEST METHOD FOR RESISTANCE
OF COMPACTED HOT MIX ASPHALT (HMA) MOISTURE
DAMAGE ACCELERATED BY THE USE OF DEICING/ANTI-
ICING CHEMICALS**

Standard Test Method for Resistance of Compacted Hot Mix Asphalt (HMA) Moisture Damage Accelerated by the Use of Deicing/Anti-Icing Chemicals

1. Scope

1.1. This method covers preparation of specimens and the measurement of the change of diametral tensile strength resulting from the effects of conditioning in solutions of deicing/anti-icing chemicals (DIAICs) compared to conditioning in water. The results may be used to predict long-term stripping susceptibility of the HMA when subjected to DIAICs and evaluate liquid anti-stripping additives that are added to the asphalt binder or pulverulent solids, such as hydrated lime or portland cement, which are added to the mineral aggregate.

1.2. The values stated in SI units are to be regarded as the standard.

1.3. *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1. ASTM Standards:

D 979 Sampling Bituminous Paving Mixtures

D 1074 Test Method for Compressive Strength of Bituminous Mixtures²

D 1559 Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus²

D 1561 Practice for Preparation of Bituminous Mixture Test Specimens by means of California Kneading Compactor²

D 2041 Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures

D 2726 Test Method for Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens²

D 3203 Test Method for Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures²

D 3387 Compaction and Shear Properties of Bituminous Mixtures by Means of the U.S. Corps of Engineers Gyrotory Testing Machine (GTM)

D 3496 Method for Preparation of Bituminous Mixture Specimens for Dynamic Modulus Testing²

D 3515 Specifications for Hot-Mixed, Hot-Laid Bituminous Paving Mixtures²

D 3549 Thickness or Height of Compacted Bituminous Paving Mixture Specimens TS-2d T 283-1 AASHTO

D 3665 Practice for Random Sampling of Construction Materials²

D 4013 Practice for Preparation of Test Specimens of Bituminous Mixtures by Means of Gyrotory Shear Compactor²

D 4123 Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures²

3. Summary of Method

3.1 *Potential for Moisture Damage Accelerated by the Use of Deicing/Anti-icing Chemicals*—The degree of susceptibility to moisture damage accelerated by the use of deicing/anti-icing chemicals is determined by preparing a set of laboratory-compacted specimens conforming to the job-mix formula without an additive. The specimens are compacted to a void content corresponding to void levels expected in the field, usually in the 6 to 8 % range. The set is divided into two subsets of approximately equal void content. One subset is conditioned in water for four days at 60°C while the other is conditioned in a solution of deicing/anti-icing chemical for four days at 60°C. The tensile strength of each subset is determined by the tensile splitting test. The potential for moisture damage accelerated by the use of deicing/anti-icing chemicals is indicated by the percent loss (or gain) in tensile strength of the subset conditioned in deicing/anti-icing solution relative to the tensile strength of the subset conditioned in water.

3.2 *Additive Effect*—The effect of an antistripping additive is determined on a set of specimens containing an additive prepared and tested as described in 3.1. The effect of an additive dosage may be estimated by repeating the tests on sets with different additive dosages.

3.3 *Plant-Produced Mixtures*—The potential for moisture damage related to the use of deicing/anti-icing chemicals or the effectiveness of an additive in a plant-produced mixture is determined on specimens that are laboratory compacted to expected field-level air void content, divided into wet and dry subsets, and evaluated as described in 3.2.

4. Significance and Use

4.1 This test method can be used to test asphalt concrete mixtures in conjunction with mixture design testing to determine the potential for moisture damage accelerated by the use of deicing/anti-icing chemicals, to determine whether or not an antistripping additive is effective, and to determine what dosage of an additive is needed to maximize the effectiveness. This test method can also be used to test mixtures produced in plants to determine the effectiveness of additives under the conditions imposed in the field.

5. Apparatus

5.1 To prepare and compact the specimens use apparatus from any one of the following: Test Method D 1559 or Practice D 4013.

5.2 *Container*, preferably Type F, of Test Method D 2041.

5.3 *Balance* in accordance with Test Method D 2726.

5.3 *Water Baths* Three:

5.3.1 One water bath in accordance with Test Method D 2726.

5.3.2. One water bath capable of maintaining a temperature of $60 \pm 1.0^{\circ}\text{C}$ ($140 \pm 1.8^{\circ}\text{F}$) for 24 h, and

5.3.3 One water bath capable of maintaining a temperature of $25 \pm 1.0^{\circ}\text{C}$ ($77 \pm 1.8^{\circ}\text{F}$).

5.4 *Loading jack and ring dynamometer* in accordance with Test Method D 1559, or a Mechanical or Hydraulic Testing Machine capable of maintaining the required strain rate and measuring load with equal or better precision.

5.5 *Loading Strips* in accordance with Test method D 4123.

6. Preparation of Laboratory Test Specimens

6.1 Make at least six specimens for each test, half to be tested after conditioning in water and the other half to be tested after conditioning in a solution of the selected deicing/anti-icing chemical.

6.2 Specimens 100 mm (4 in.) in diameter by 63.5 ± 2.5 mm (2.5 ± 0.1 in.) thick, or 150 mm (6 in.) in diameter by 95 ± 5 mm (3.75 ± 0.20 in.) thick are used. Specimens 150 mm (6 in.) in diameter by 95 ± 5 mm (3.75 ± 0.20 in.) thick should be used if aggregate larger than 25 mm (1 in.) is present in the mixture.

6.3 Prepare mixtures in batches large enough to make at least three specimens or, alternatively, prepare a batch large enough to just make one specimen at a time. If preparing a multi-specimen batch, split the batch into single-specimen quantities before placing in the oven.

6.4 When a liquid antistripping additive is used, heat a sufficient quantity of asphalt cement for one batch to $150 \pm 6^\circ\text{C}$ ($300 \pm 10^\circ\text{F}$) in a closed 1 L (1-qt) can in an oven. Add the required quantity of additive and immediately mix, for approximately 2 min, with a mechanical stirrer approximately 25 mm (1 in.) from the bottom of the container. Maintain the treated asphalt cement at $150 \pm 6^\circ\text{C}$ ($300 \pm 10^\circ\text{F}$) in the closed can until it is used. Discard the treated asphalt cement if not used the same day it is prepared, or if allowed to cool so that it requires reheating.

6.5 When using a pulverulent solid antistripping additive, use the addition procedure simulating the procedure expected in the field. Follow the procedure specified in either 6.5.1, 6.5.2, or 6.5.3.

6.5.1 When dry powder is added to dry aggregate, dry, batch, and heat the mineral aggregate to $150 \pm 6^\circ\text{C}$ ($300 \pm 10^\circ\text{F}$). Add the required quantity of additive to the aggregate, and thoroughly mix the entire mass until a uniform distribution of additive is achieved. Take care to minimize the loss of additive to the atmosphere in the form of dust. After mixing, maintain the treated aggregate at the required mixing temperature until it is used.

6.5.2 When dry powder is added to damp aggregate, batch the damp mineral aggregate, and adjust the moisture content of the combined aggregate to the expected field moisture level. Add the required quantity of additive to the damp aggregate, and thoroughly mix the entire mass until a uniform distribution of additive is achieved. Take care to minimize the loss of additive to the atmosphere in the form of dust. After mixing, dry the treated aggregate, heat to the required mixing temperature, and maintain at the temperature until it is used.

6.5.3 When powder slurry is used, add the required quantity of additive to water using the powder to water ratio expected in the field. Take care to minimize the loss of additive to the atmosphere in the form of dust. To prevent settling, continuously mix the resulting slurry until it is used. Batch the damp mineral aggregate, adjust the moisture content as required 6.5.2, add the required quantity of slurry, and thoroughly mix the entire mass until a uniform distribution of slurry is achieved. After mixing, dry the treated aggregate, heat to the required mixing temperature, and maintain at that temperature until used.

6.6 Proportion, mix and compact specimens in accordance with one of the following: Test Method D 1559 or Practice D 4013. If Test Method D 1559 is used, either a manual or mechanical hammer may be used.

6.6.1 After mixing, stabilize the mixture temperature of each specimen at the required compaction temperature, in a closed container, in an oven for 1 to 2 h. If preparing a multi-specimen batch, split the batch into single-specimen quantities before placing into the oven.

6.6.2 Compact the specimens to 7 ± 1 % air voids, or a void level expected in the field at the time of construction. This void level can be obtained by adjusting the following: The number of blows in a marshall hammer compaction or the number of gyrations in a gyratory compactor.

6.6.3 Cool specimens in the mold to room temperature as rapidly as possible in a stream of moving air, extract from molds, then follow the procedure outlined in Section 8 within 24 h.

7. Preparation of Field Specimens

7.1 Select a truck to be sample in accordance with Practice D 3665.

7.2 Secure a sample from the truck at the plant in accordance with Practice D 979.

7.3 Stabilize the mixture temperature to approximately the temperature found in the field when rolling begins. Maintain this temperature in a closed container, in an oven if necessary, for approximately the time lapse between mixing and the start of actual rolling.

7.4 Compact the specimens in accordance with 6.6.2, and cool and extract from the molds in accordance with 6.6.3.

7.5 If specimens are not to be compacted in the field laboratory, place the samples in a sealed container, transport to the laboratory, and reheat to the temperature required in 7.3. Proceed with the steps in 7.4.

8. Procedure

8.1 Determine the theoretical maximum specific gravity in accordance with Test Method D 2041.

8.2 Determine the specimen height in accordance with Test Method D 3549.

8.3 determine the bulk specific gravity in accordance with Test Method D 2726, and express the volume of the specimen in cubic centimeters. The term ($B-C$) in Test method D 2726 is the volume of the specimen in cubic centimeters.

8.4 Calculate the percent air voids in accordance with Test Method D 3203, and express the volume of air in cubic centimeters. The volume of air is the volume of the specimen in 8.3 multiplied by the percent air voids.

8.5 Sort the specimens into two subsets so that the average air voids of the two subsets are approximately equal.

8.6 Condition one subset of specimens in distilled water at $60 \pm 1^\circ\text{C}$ ($140 \pm 1.8^\circ\text{F}$) for 4 days (96 h.).

8.7 Condition the other subset of specimens in a 2 % solution of the selected deicing/anti-icing chemical at $60 \pm 1^\circ\text{C}$ ($140 \pm 1.8^\circ\text{F}$) for 4 days (96 h.).

8.8 Adjust the temperature of the specimens from both subsets by soaking in a water bath for 1 h at $25 \pm 1^\circ\text{C}$ ($77 \pm 1.8^\circ\text{F}$).

8.9 Determine the tensile strength at $25 \pm 1^\circ\text{C}$ ($77 \pm 1.8^\circ\text{F}$) of both subsets.

8.9.1 Place a specimen into the loading apparatus and position the loading strips so that they are parallel and centered on the vertical diametral plane. Apply a diametral load at 50 mm/min (2 in./min) until the maximum load is reached, and record the maximum load.

8.9.2 Continue loading until the specimen fractures. Break the specimen open and visually estimate and record the approximate degree of moisture damage, if any.

8.9.3 Inspect all surfaces, including the failed faces, for evidence of cracked or broken aggregate, that may influence test results, and record observations.

9. Calculation

9.1 calculate the tensile strength as follows:

$$S_t = 2000P/\pi D \text{ (kPa)}$$

Or

$$S_t = 2P/\pi D \text{ (psi)}$$

Where:

S_t = tensile strength kPa (psi)

P = maximum load, N (lbf)

T = specimen height immediately before tensile test, mm (in.), and

D = specimen diameter, mm (in.)

9.2 Calculate the retained tensile strength/deicer treatment:

$$TSR / D = S_{td} / S_{tw} 100$$

Where:

TSR/D = retained tensile strength/deicer treatment, %

S_{td} = average tensile strength of the specimens conditioned in deicing/anti-icing solution, kPa (psi), and

S_{tw} = average tensile strength of the specimens conditioned in water, kPa (psi)

10. Report

10.1. Report the following information:

10.1.1 Number of specimens in each subset,

10.1.2 Average air voids of each subset,

10.1.3 Average swell after conditioning of each subset,

10.1.4 Tensile strength of each specimen in each subset,

10.1.5 Tensile strength ratio,

10.1.6 Results of visually estimated moisture damage observed when the specimen fractures, and

10.1.7 Results of observations of cracked or broken aggregate.