GUIDE TO THE PREVENTION AND RESTORATION OF EARLY JOINT DETERIORATION IN CONCRETE PAVEMENTS

DECEMBER 2016

Iowa State University
Institute for Transportation

National Concrete Pavement Technology Center

Tech Center

IOWA DOT

Cover photo: New pavement constructed in fall 2015 in West Des Moines, Iowa, with attention to the research and principles related to durable concrete joints discussed in this guide (photo courtesy of Ben McAlister, City of West Des Moines)

Technical Report Documentation Page

1. Report No.	**2. Government Accession No.**	**3. Recipient's Catalog No.**	
IHRB Project TR-697			
4. Title Guide to the Prevention and Restoration of Early Joint Deterioration in Concrete Pavements		**5. Report Date** December 2016	
		6. Performing Organization Code	
7. Author(s) Jason Weiss (orcid.org/0000-0003-2859-7980), M. Tyler Ley (orcid.org/0000-0002-8782-2028), Larry Sutter (orcid.org/0000-0002-4047-3094), Dale Harrington (orcid.org/0000-0003-0912-2306), Jerod Gross (orcid.org/0000-0003-2283-5845), and Steven Tritsch (orcid.org/0000-0002-2938-5915)		**8. Performing Organization Report No.** InTrans Project 15-555	
9. Performing Organization Name and Address National Concrete Pavement Technology Center Iowa State University 2711 South Loop Drive, Suite 4700 Ames, IA 50010-8664		**10. Work Unit No. (TRAIS)**	
		11. Contract or Grant No.	
12. Sponsoring Organization Name and Address Iowa Highway Research Board Iowa Department of Transportation 800 Lincoln Way Ames, IA 50010		**13. Type of Report and Period Covered** Final Report and Guide	
		14. Sponsoring Agency Code IHRB Project TR-697	
15. Supplementary Notes Visit www.cptechcenter.org for color pdfs of this and other research reports.			
16. Abstract In recent years, premature joint deterioration has occurred in concrete pavements in snowbelt states. Research has found two primary mechanisms behind the deterioration: (1) Certain deicing/anti-icing salts—calcium chloride and magnesium chloride—react with cement paste to form calcium oxychloride, an expansive material that is detrimental to concrete pavement performance (sodium chloride is not as reactive). (2) Freeze-thaw damage occurs in joints when a critical degree of saturation is reached or exceeded; salts can exacerbate the problem by keeping joints in a high state of saturation. As a result of these findings, the Iowa Highway Research Board commissioned the National Concrete Pavement Technology Center to write this guide. The primary goal is to help Iowa's concrete pavement engineers understand the causes of premature joint deterioration, focusing on deterioration due to salt reactivity and joint saturation. The guide also discusses strategies for preventing or limiting such deterioration. One strategy is to limit applications of calcium chloride and magnesium chloride in order to mitigate the formation of calcium oxychloride. Another critical strategy is to keep the saturation level of the concrete below approximately 85 percent. This can be accomplished by designing a durable concrete that includes an adequate air-void system, supplementary cementitious materials, and a low water-to-cementitious-materials ratio, as well as by providing good drainage to the pavement system and following other best practices in concrete pavement design, construction, and maintenance. This guide also provides a summary of joint repair and restoration strategies, including a decision flow chart. Finally, the guide offers guidelines for developing project specifications, with appropriate references to Iowa DOT Standard Road Plans, Standard Specifications, and Instructional Memoranda.			
17. Key Words concrete pavement—critical degree of saturation—freeze-thaw damage—joint deterioration—joint maintenance—joint restoration—pavement durability—pavement specifications		**18. Distribution Statement** No restrictions.	
19. Security Classification (of this report) Unclassified.	**20. Security Classification (of this page)** Unclassified.	**21. No. of Pages** 39	**22. Price** NA

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December 2016

AUTHORS

Jason Weiss, Ph.D., Oregon State University

M. Tyler Ley, Ph.D., P.E., Oklahoma State University

Larry Sutter, Ph.D., Michigan Tech University

Dale Harrington, P.E., Snyder and Associates, Inc.

Jerod Gross, P.E., Snyder and Associates, Inc.

Steven L. Tritsch, P.E., National Concrete Pavement Technology Center, Iowa State University

PRINCIPAL INVESTIGATOR

Steven L. Tritsch, P.E., National Concrete Pavement Technology Center, Iowa State University

MANAGING EDITOR

Marcia Brink, National Concrete Pavement Technology Center, Iowa State University

COPYEDITOR

Carol Gostele, Birch Tree Editing

ABOUT THIS MANUAL

This guide is a product of the National Concrete Pavement Technology Center (CP Tech Center) at Iowa State University. Its goal is to help Iowa's transportation agencies and contractors benefit from the latest research about the causes and prevention of premature joint distress when they are designing and building concrete pavements.

ACKNOWLEDGMENTS

The CP Tech Center and the authors would like to thank the Iowa Department of Transportation and the Iowa Highway Research Board (TR-697) for sponsoring this project. In addition, we appreciate the members of the technical advisory committee, who took time from their busy schedules to review drafts of this document and provide valuable feedback.

Matthew Cox, P.E., City of Council Bluffs
Todd Hanson, P.E., Iowa Department of Transportation
Sarah Okerlund, P.E., City of Ankeny
Gordon Smith, P.E., Iowa Concrete Paving Association
Dr. Peter Taylor, P.E., National Concrete Pavement Technology Center
Jacob Thorius, P.E., Washington County
Bob Younie, P.E., Iowa Department of Transportation

MISSION

The mission of the National Concrete Pavement Technology Center is to unite key transportation stakeholders around the central goal of advancing concrete pavement technology through research, technology transfer, and technology implementation.

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NONDISCRIMINATION STATEMENT

IOWA DEPARTMENT OF TRANSPORTATION STATEMENTS

The preparation of this report was financed in part through funds provided by the Iowa Department of Transportation through its "Second Revised Agreement for the Management of Research Conducted by Iowa State University for the Iowa Department of Transportation" and its amendments.

FOR MORE INFORMATION

Peter Taylor, Director
Marcia Brink, Senior Editor
National Concrete Pavement Technology Center
Iowa State University Research Park
2711 S. Loop Drive, Suite 4700
Ames, IA 50010-8664
515-294-9480
www.cptechcenter.org/
mbrink@iastate.edu

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EXECUTIVE SUMMARY

For more than a century, portland cement concrete pavements have served the traveling public very well. During this time, joint design has undergone many changes in regard to spacing, width, depth, and formation. In addition, materials engineers have gained knowledge about the concrete mixture itself related to improving joint performance.

In recent years, premature concrete pavement joint distresses have occurred, primarily affecting the mortar in the joints. This premature deterioration has occurred not only in Iowa but in many snowbelt states in conjunction with the use of more aggressive winter maintenance practices such as deicing and anti-icing. Recent research (Weiss and Farnam 2015) into the premature deterioration phenomenon has found two primary mechanisms behind the deterioration: (1) Certain deicing/anti-icing salts—calcium chloride and magnesium chloride—react with cement paste to form calcium oxychloride, an expansive compound that is detrimental to concrete pavement performance. (2) Freeze-thaw damage is occurring in joints when a critical degree of saturation is reached or exceeded. Salts can exacerbate the problem by keeping joints in a high state of saturation.

As a result of the research findings, the Iowa Highway Research Board commissioned the National Concrete Pavement Technology Center to write this guide. The primary goal is to help Iowa's concrete pavement engineers understand the causes of premature joint deterioration, focusing on deterioration due to deicing/anti-icing salts and joint saturation.

In addition, the guide discusses strategies for preventing or limiting such deterioration, which are summarized here:

- Limit applications of calcium chloride and magnesium chloride for winter deicing and anti-icing activities. (Sodium chloride is much less reactive.)
- Keep the saturation level of the concrete below approximately 85 percent for improved freeze-thaw durability. This can be accomplished by designing a durable concrete that includes
    - An adequate air-void system (consider an air content between 5 and 8 percent, with 5 percent minimum in place)
    - Supplementary cementitious materials (consider a cement replacement rate of 20–25 percent Class F fly ash or 30–35 percent Class C fly ash or a combination of 20 percent slag and 20 percent Class C fly ash)
    - Low water-to-cementitious-materials ratio (consider a w/c ratio of 0.40)
    - Well graded aggregates

This guide also provides summaries of joint maintenance, repair, and restoration strategies, including a decision matrix. Finally, the guide offers guidelines for developing project specifications, with appropriate links to Iowa Department of Transportation Standard Road Plans, Standard Specifications, and Instructional Memoranda.

CHAPTER 1. BRIEF HISTORY OF CONCRETE PAVEMENT JOINT DESIGN AND PERFORMANCE IN IOWA

Since Iowa's first portland cement concrete pavement was constructed in Le Mars in 1904 (Hanson 2009), engineers have observed joints from a design as well as a performance standpoint. Many concrete pavements built in Iowa with durable materials and good construction techniques have performed well beyond their estimated design lives. Two examples are the Eddyville cemetery road placed in 1909 and still in service today (Figure 1-1) and a 16-foot (ft) wide pavement with 25-ft transverse joints near Moscow, Iowa, which was placed in 1914 and is in service today as a bike trail (Figure 1-2).

In the last 100-plus years, Iowa Department of Transportation (DOT) engineers have adjusted concrete pavement designs to fit various needs, from 18-ft-wide pavements with transverse joints every 12 ft to 100 ft to continuously reinforced concrete pavements (CRCPs) with no contraction joints. In the early years, joints were formed by scoring or with wood, steel, and/or bituminous strips; now they are formed by wet or dry sawing.

Between 1920 and 1925, no joints were placed in 18-ft-wide pavements. From 1925 to 1930, a longitudinal joint was installed by steel or mastic, but still no transverse joints were placed. Beginning in 1930, premolded bituminous expansion joints were placed every 80 ft to 100 ft, with temperature steel (mesh reinforcement) to hold the uncontrolled cracks together. In 1935, pavement widths on state highways jumped to 20 ft, and this became standard practice until 1949 when 22-ft pavements became the norm on the primary system through 1971, with 24-ft pavements entering the market in 1960 on selected routes. In 1958 the standard for transverse joint spacing was established at 20 ft. Skewed joints were specified from 1980 to 2005 to assist in aggregate interlock load transfer efficiency.

Pavements for the interstate system have always been a minimum of 24 ft wide, with tiebars in the longitudinal joint. From 1958 to 1965, bar mat reinforcing was the standard with doweled contraction joints at 76.5-ft spacing. During a 10-year period from 1966 to 1976, CRCPs were built on selected interstate projects to eliminate the need for transverse contraction joints. Since 1976, 20-ft transverse spacing has been the standard for all interstate work. Twenty-six-ft-wide lanes were first constructed in 1988 on Iowa's interstate system.

Today the Iowa DOT's typical concrete pavement section is 26 ft wide with transverse joint spacing of 20 ft and dowels for pavements 8 inches (in.) thick or more.

Materials have changed as well, often in response to specific challenges. For example, beginning in the early 1900s materials engineers faced durability issues in some concrete pavements containing gravel contaminated with unsound materials such as shale, coal, iron, and chert. As a result, specifications were implemented to set limits on unsound materials. In the late 1930s, gravel gave way to limestone as the established coarse aggregate for Iowa's concrete pavements. About that time, some pavements containing aggregate from certain limestone formations were experiencing significant D-cracking issues. Extensive research was done to identify questionable coarse aggregate sources, resulting in a process for identifying aggregate durability in regard to freeze-thaw cycles. In 1952 air entrainment was required for the first time to help mitigate freeze-thaw damage.

In 1991 a section of U.S. 20 in Hamilton and Webster counties, which was placed in 1987, began to show visible deterioration at the joints and along vibrator trails (Figure 1-3). Similar deterioration began to appear on several other paving projects placed between 1986 and 1994. Several research projects identified the cause as a loss of air at the joints and in the vibrator trails, increasing the pavements' exposure

Figure 1-1. Eddyville, Iowa, cemetery road in spring 2016 (photo courtesy of Kevin Merryman, Iowa DOT)

Figure 1-2. Moscow road in 2014 (photo courtesy of Kurt Smith, Applied Pavement Technology, Inc.)

to freeze-thaw damage. Solutions were also identified to avoid similar problems in new pavements: require vibration monitoring equipment to ensure proper vibration speed and control, improve workability through optimized aggregate gradations, and increase the use of slag cement in the mixture.

More recently, premature joint deterioration (that is, generally occurring within 10 years of construction) has been noted in a few concrete pavements, even in pavements containing durable freeze-thaw resistant aggregates and entrained air (Figure 1-4). An informal survey found that other areas of the country, particularly snowbelt states, were experiencing the same problem. The deterioration was generally limited to areas that used salt during the winter months to mitigate the formation and/or bonding of ice on the pavement surface. Although the occurrences were limited, Iowa and other affected states were eager to determine the exact cause.

As discussed in the following chapter, subsequent research has found that premature joint deterioration has occurred (1) in pavements with high moisture content (at or above a critical degree of saturation) and (2) where certain deicing/anti-icing salts have reacted with components of the paste, resulting in a loss of material in and around the aggregate at the joint and crack face.

The primary goal of this guide is to help designers, materials engineers, and paving contractors understand the mechanisms behind premature joint deterioration, particularly as the mechanisms are related to saturated joints and the use of deicing/anti-icing salts. This guide also outlines best practices in design, mixtures/materials, and construction, as well as routine maintenance, to prevent or mitigate joint deterioration. It also provides information about repairing or reconstructing damaged joints. Finally, the guide offers guidelines for developing project specifications related to preventing or repairing joint deterioration.

Note that, although this guide's focus is on joints, much of the information herein is relevant for constructing and maintaining durable concrete pavements in general.

Figure 1-3. Vibrator trails

Figure 1-4. Joint deterioration (Photo courtesy of Jim Grove, FHWA)

CHAPTER 2. TYPES AND MECHANISMS OF JOINT DETERIORATION

In recent years there has been an increase in reports of premature deterioration around joints in concrete pavements (Sutter et al. 2006; Shi et al. 2009; Taylor et al. 2012; Jones et al. 2013). Such deterioration has been observed in some concrete pavements even when freeze-thaw resistant aggregates and air-entrainment systems have been used. Two factors have been identified as the primary causes of the damage: (1) saturation that leads to classic freeze-thaw damage, and (2) a reaction between certain salts and the matrix of concrete (Weiss et al. 2011; Suraneni et al. 2016b). Joints are particularly susceptible to these two causes, because water and deicing solution can collect in the joints. This chapter provides a short overview of the mechanisms associated with each form of damage and then discusses potential aspects of constituent material composition, mixture composition, and construction practices that can be employed to reduce the potential for damage.

Saturated Freeze-thaw Damage

Concrete is susceptible to freeze-thaw damage when it reaches a critical degree of saturation (DOS) (Fagerlund 1977; Li et al. 2011).

Critical degree of saturation

For degrees of saturation below the critical DOS (e.g., 85 percent in Figure 2-1), freeze-thaw damage is generally not observed to occur even after a large number of freeze-thaw cycles. As soon as the DOS exceeds the critical DOS (in this case 85 percent), damage initiates in the material, regardless of the quantity and quality of the air system. This implies that once the critical DOS is reached, even a single freezing cycle can result in damage. One role of air is to reduce the DOS and enable the pressures generated during ice formation to be reduced.

Sorption-based models have been proposed (Fagerlund 1977; Bentz et al. 2001; Yang et al. 2006; Todak et al. 2015a) to predict the time required for a concrete sample to reach critical DOS after being continuously exposed to water. It is assumed that once the concrete reaches the critical DOS, freeze-thaw damage occurs. The sorption model is illustrated in Figure 2-2. The model describes the relationship between the DOS in the sample and the critical DOS (shown as the yellow region above the dashed line at 85 percent DOS) that would result in freeze-thaw damage. The initial absorption occurs very quickly (during the first 12 to 24 hours for 50 millimeter [mm] samples) and is represented by the blue line. The initial absorption (also known as the matrix saturation, which is shown as the red dot) can be predicted using simple volumetric mixture proportioning concepts (Todak et al. 2015b; Weiss 2016). This initial absorption describes the filling of the gel and smaller capillary pores. After the gel and capillary pores are filled, entrained and entrapped pores will fill over a longer time (this is known as secondary sorptivity or absorption). This secondary sorptivity is shown by the red line. If the entrained and entrapped voids fill in to a point where the concrete exceeds the critical DOS, the concrete will begin to show damage. It should also be noted that the critical DOS can vary (from 78 to 91 percent) depending on the quality of the air-void system.

Entrained air can improve freeze-thaw durability by reducing the DOS in concrete (Li et al. 2011, 2016). Air-entraining admixtures stabilize air bubbles generated during mixing to add large (approximately 0.002 to 0.050 in.), stable, air-filled voids to the paste portion of the concrete (Mindess and Young 1981). These entrained air voids provide air-filled voids that reduce the initial DOS, as shown in Figure 2-3 (i.e., the initial saturation is the saturation that occurs during the first 24 hours, as denoted by the nick point or transition from the

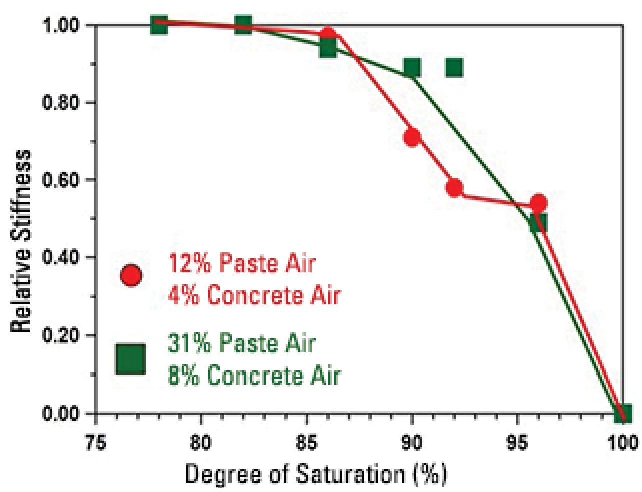


Figure 2-1. Influence of the DOS on stiffness degradation due to freeze-thaw damage (Li et al. 2011)

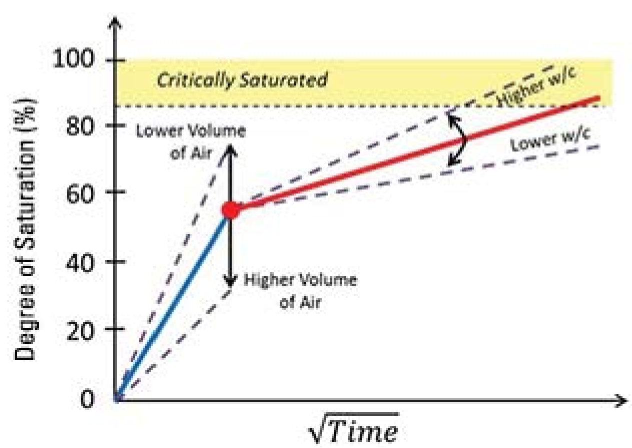
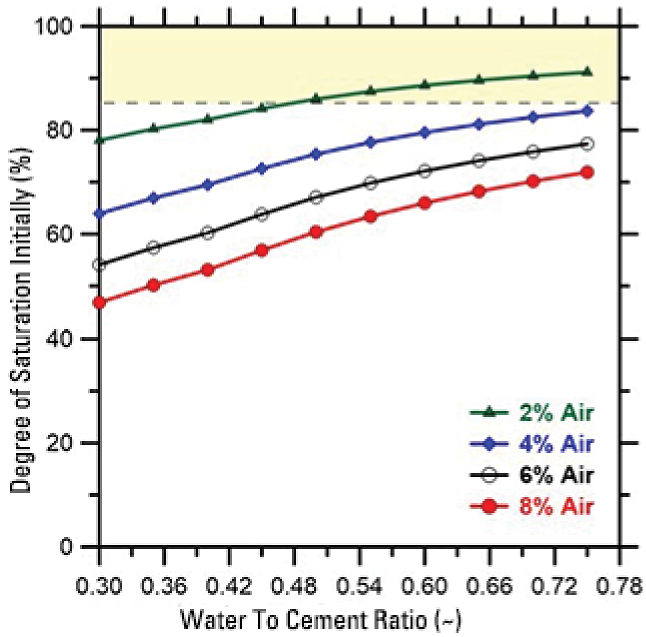


Figure 2-2. A conceptual illustration of the sorption-based freeze-thaw model

blue to the red line in Figure 2-2). Figure 2-3(a) shows that both air content and water-to-cement (w/c) ratio contribute to the initial DOS when the same amount of cement is used in a mixture. As the w/c ratio increases and air content decreases, the concrete is closer to critical saturation after only a short time of exposure to water. The mixture composition can also impact the slope of the secondary absorption (red line in Figure 2-2), with data shown in Figure 2-3(b) for a straight cement mixture. Preliminary data have shown that this can be substantially reduced for mixtures with supplementary cementitious materials. More data are needed, however, to fully understand the impact of all changes in a mixture composition on this behavior.

(a)



(b)

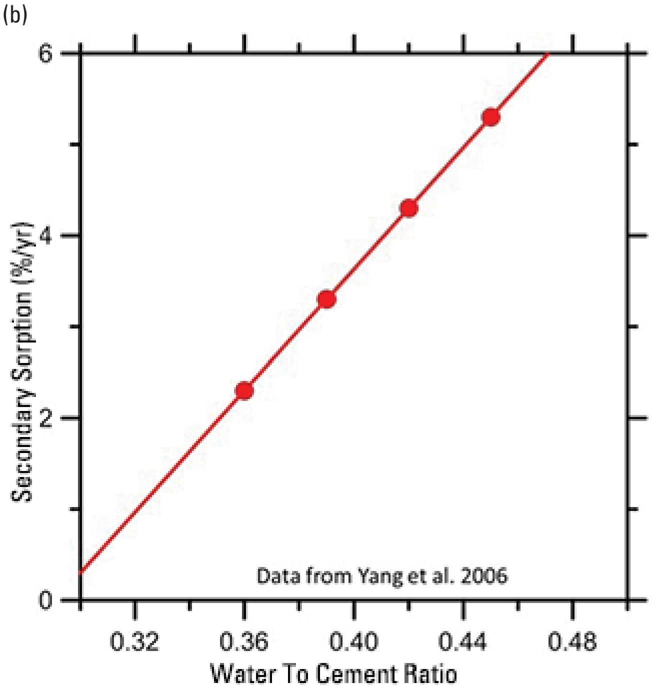


Figure 2-3. Influence of (a) the w/c ratio and air content on SMatrix and (b) the w/c ratio on the secondary sorption (Todak et al. 2015b)

To illustrate the interrelation between air content and w/c ratio on the freeze-thaw performance of concrete, a sorption-based model was developed that incorporated material variability using the Monte Carlo method (Weiss 2016). The base concrete was a typical concrete mixture with a w/c ratio of 0.42, a designed air content of 6 percent, 564 pounds per cubed yard (lb/yd^3) of cement, 1,700 lb/yd^3 of coarse aggregate with a specific gravity of 2.68, and 1,467 lb/yd^3 of fine aggregate with a specific gravity of 2.75. The model also assumes that the concrete is in continual contact with water (i.e., there is no drying), which may be the case for pavements with poor drainage or trapped water at the joints. Figure 2-4(a) was formulated by varying the w/c ratio and fine aggregate to create a mixture to yield a cubic yard of concrete (Weiss 2016). While additional data are needed to build the database of material properties used in the model, two trends can be noticed: (1) increasing the air content reduces the initial degree of saturation, requiring a longer time for the concrete to reach critical saturation; and (2) increasing the w/c ratio increases the transport of water through concrete, resulting in more rapid saturation.

Variability in the concrete mixture can impact the life-cycle performance of the concrete. The Monte Carlo simulation model was used to indicate the influence of variation in w/c ratio (fixed at 5 percent coefficient of variation, COV) and variation in air content (5 percent, 15 percent, and 25 percent COV). The variation in the air content corresponds to 95 percent of the data ranging between 5.4 percent and 6.6 percent for the case with 5 percent COV, 4.2 to 7.8 percent for 15 percent COV, and 3.0 to 9.0 percent for the 25 percent COV. It can be noted from Figure 2-4(b) that the deterministic prediction of the time to saturation is similar to the time predicted for 50 percent of the elements to fail. It is interesting to note, however, that higher variability in air content (5 to 15 to 25 percent COV) results in time of failure of 17.2, 13.8, and 7.9 years, respectively. This would imply that despite the average air content, which is the same for each of these cases, repairs would need to be performed at an earlier age for systems with a higher variability. This clearly indicates the importance of controlling the variation in the air content during production. This needs to be balanced, however, with an understanding of how accurately the air content can be measured (Pellinen et al. 2005).

Resulting damage

The damage associated with fluid saturation and classic freeze-thaw generally manifests as cracks that are oriented parallel to the joints and that occur within approximately 4 to 6 in. of the walls of the original joint. These cracks develop in response to spalling that begins at the base of the saw cuts used to construct the joints.

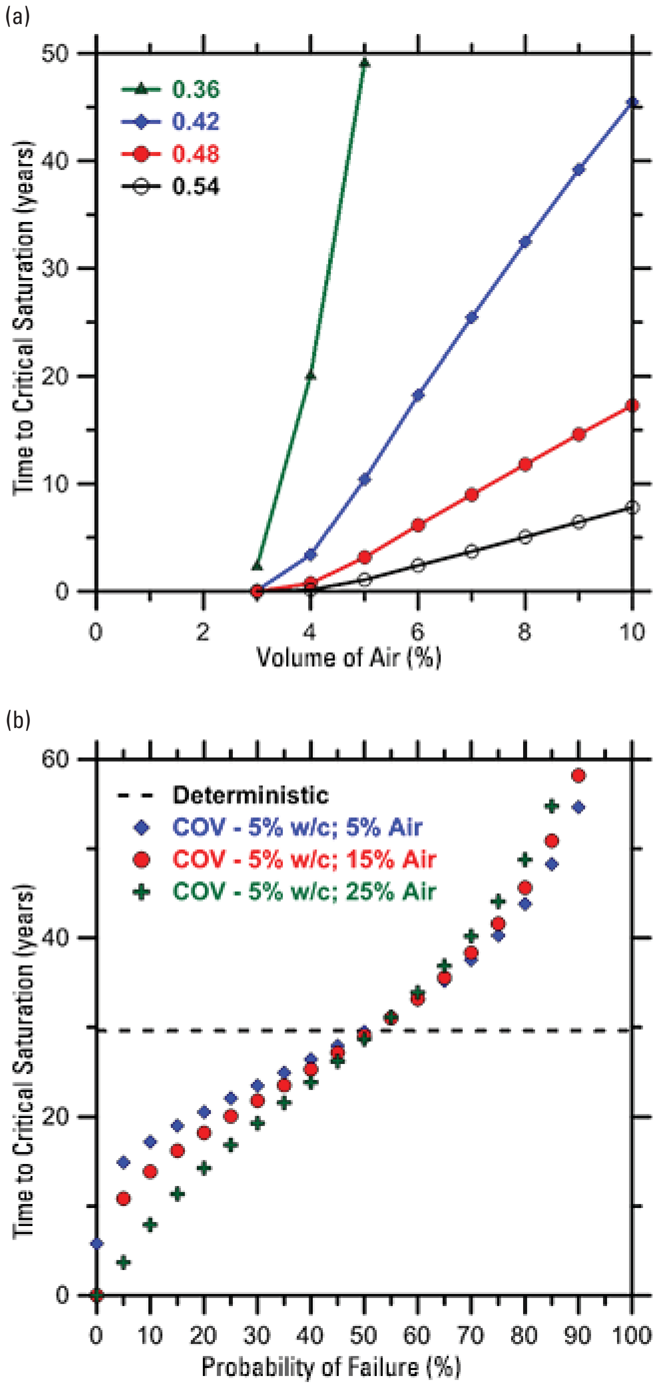


Figure 2-4. The influence of w/c ratio and air volume on the time required for 20 percent of the concrete to reach critical saturation (assuming 5 percent COV in w/c ratio and 15 percent COV in air content)

Potential mitigation methods

While the previous section has focused on the volume of air (quantity of air voids), both the specific volume of air (e.g., 5 percent) in a mixture and the spacing of that air (e.g., a critical spacing factor of 0.008 in.) influence the performance. Current approaches to design specify mixtures that utilize w/c ratio, the volume of air voids, and a measure of air-void spacing to ensure quality. It is anticipated that future approaches will replace w/c ratio with formation factor (F factor), and the air quality and quantity may be replaced with measures like the super air meter (SAM) number. These tests (F factor as assessed through electrical resistivity, and air properties as assessed through SAM) have the advantage in that they may be used in daily quality-control operations.

Calcium Oxychloride Formation, Expansion, and Damage

High concentrations of salts can react with cementitious matrices, resulting in secondary deposits in concrete that can diminish durability. In some areas of the country, very high concentrations of salts are applied to pavements because they are believed to be more effective in ice removal, or deicing. High salt concentrations are also a common result of anti-icing practices, where deicer solutions are applied to dry, rather than snow- and ice-covered, pavements. Even when low concentrations of deicers are applied to pavements, the evaporation that occurs during drying cycles will eventually produce a highly concentrated deicing solution. At some point, these concentrations will reach a level of supersaturation that drives the precipitation of secondary mineral deposits in the concrete.

Chemical reactions

The first type of reaction that may be observed occurs between the chlorides in the deicing salt and the aluminate phases present in concrete to form Freidel's or Kuzel's salts. Note that while the Freidel's and Kuzel's salt reactions can occur over a wide range of concentrations and temperatures, they are generally not considered to be very detrimental to the performance of concrete other than reducing the space available for fluid movement into and out of air voids, which would increase the DOS (Farnam et al. 2015c).

The reaction for these salts is shown in Equations 1–3: Calcium chloride ($CaCl_2$) reacts with tricalcium aluminate ($3CaO\ Al_2O_3$) from the binder to form Freidel's salt ($3CaO{\cdot}Al_2O_3{\cdot}CaCl_2{\cdot}10H_2O$) (Eqn 1). Similarly, calcium chloride can react with the hydration product monosulfate ($3CaO\ Al_2O_3\ CaSO_4\ 12H_2O$) to form Freidel's salt and gypsum (Eqn 2). Calcium monosulfoaluminate hydrate ($3CaO{\cdot}Al_2O_3{\cdot}CaSO_4{\cdot}12H_2O$) can be converted to Kuzel's salt ($3CaO{\cdot}Al_2O_3{\cdot}0.5CaSO_4{\cdot}0.5CaCl_2{\cdot}11H_2O$), ettringite ($3CaO{\cdot}Al_2O_3{\cdot}3CaSO_4{\cdot}32H_2O$), and aluminum hydroxide ($2Al_2O_3{\cdot}3H_2O{\cdot}6OH$) (Mesabah et al. 2012).

Freidel's Salt ($3CaO \bullet Al_2O_3 \bullet CaCl_2 \bullet 10H_2O$)

$$CaCl_2 + 3CaO \bullet Al_2O_3 + 10H_2O \rightarrow 3CaO \bullet Al_2O_3 \bullet CaCl_2 \bullet 10H_2O \qquad \text{Eqn 1}$$

$$CaCl_2 + 3CaO \bullet Al_2O_3 \bullet CaSO_4 \bullet 12H_2O \rightarrow 3CaO \bullet Al_2O_3 \bullet CaCl_2 \bullet 10H_2O + CaSO_4 \bullet 2H_2O \qquad \text{Eqn 2}$$

Kuzel's salt ($3CaO \cdot Al_2O_3 \cdot 0.5CaSO_4 \cdot 0.5CaCl_2 \cdot 11H_2O$)

$$3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O + 6Cl + 28H_2O \rightarrow 3CaO \cdot Al_2O_3 \cdot 0.5CaSO_4 \cdot 0.5CaCl_2 \cdot 11H_2O + 2[3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O] + 2Al_2O_3 \cdot 3H_2O \cdot 6OH \quad \text{Eqn 3}$$

The second type of reaction that may be observed, and that can cause more damage to concrete, occurs between calcium chloride and calcium hydroxide ($Ca[OH]_2$) (CH in cement chemistry shorthand) to result in calcium oxychloride ($CaCl_2 \cdot 3Ca[OH]_2 \cdot 12H_2O$), as shown in Equation 4. Calcium oxychloride is expansive and can result in damage to the cement matrix. Greater amounts of calcium oxychloride form at higher concentrations (Farnam et al. 2014; Farnam et al. 2015a) and at temperatures greater than freezing, as shown in Figure 2-5 (Farnam et al. 2016).

Calcium Oxychloride ($CaCl_2 \cdot 3Ca(OH)_2 \cdot 12H_2O$)

$$3Ca(OH)_2 + CaCl_2 + 12H_2O \rightarrow CaCl_2 \cdot 3Ca(OH)_2 \cdot 12H_2O \quad \text{Eqn 4}$$

While the previous paragraphs have focused on calcium chloride deicing salt, it should be noted that magnesium chloride ($MgCl_2$) deicing salt can also produce calcium oxychloride. The magnesium chloride will first react to form Brucite ($Mg[OH]_2$) (Eqn 5a). The magnesium will exchange with the calcium (Eqns 5b and 5c), resulting in a reaction similar to that shown for $CaCl_2$ (Eqn 4). Although the temperature for magnesium oxychloride formation is approximately 15°C greater than for calcium oxychloride formation (Farnam 2015d), samples exposed to $MgCl_2$ show both magnesium oxychloride and calcium oxychloride formation.

$$Ca(OH)_2 + MgCl_2 \rightarrow CaCl_2 + Mg(OH)_2 \quad \text{Eqn 5A}$$

$$(3 \text{ or } 5)Mg(OH)_2 + MgCl_2 + 8H_2O \rightarrow (3 \text{ or } 5)Mg(OH)_2 \cdot MgCl_2 \cdot 8H_2O \quad \text{Eqn 5B}$$

$$3Ca(OH)_2 + CaCl_2 + 12H_2O \rightarrow 3Ca(OH)_2 \cdot CaCl_2 \cdot 12H_2O \quad \text{Eqn 5C}$$

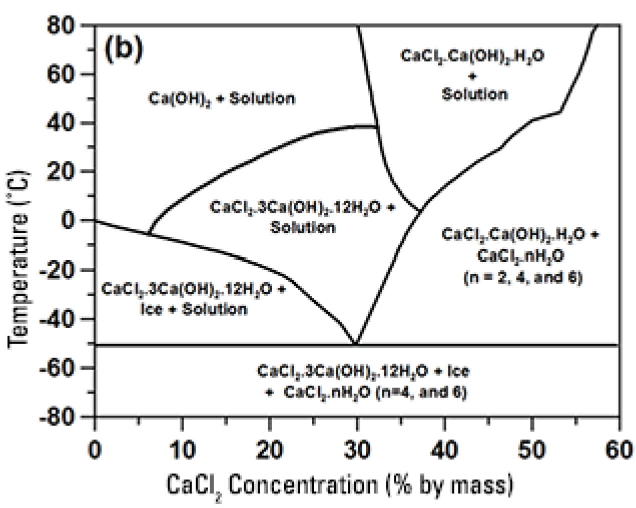


Figure 2-5. Phase diagrams illustrating the temperature for calcium oxychloride formation when hydrated cement paste is brought into contact with calcium chloride solution (Farnam et al. 2016)

Note that sodium chloride (NaCl) is not as reactive with the cement matrix in concrete (Suraneni et al. 2016 [in press]). Some studies have shown that calcium magnesium acetate (CMA) solutions may be the most deleterious deicing chemical with respect to chemical attack of concrete (Darwin et al. 2007).

Changes in fluid transport

It is important to note that the reaction that occurs between the chlorides in the deicing salt can dramatically reduce fluid transport in concrete. This can be seen in data from Figure 2-6, where the initial rate of absorption (sorptivity) can be dramatically reduced after the formation of calcium oxychloride (approximately 12 percent $CaCl_2$ at 23°C in Figure 2-5). The reduction in initial rate of absorption in concrete with a type I cement is more gradual than in concrete with type V cement because of the formation of the Friedel's and Kuzel's salts (Villani et al. 2015). Lucero et al. (submitted 2015) demonstrated that the reduction in the rate of sorption was due to limited solution ingress due to the formation of salt in the matrix that effectively "seals off" the core of the concrete, minimizing further fluid ingress. This can also explain why chloride ingress rates are often much lower in systems that have been exposed to $CaCl_2$ or $MgCl_2$ when compared with systems exposed to sodium chloride (NaCl) (Bu et al. 2015).

Resulting damage

Damage associated with salt-matrix reactions consists of the loss of concrete "flakes" ranging from 0.125 to 0.25 in. from around the joint. This damage tends to dislodge aggregate, because calcium oxychloride preferentially occurs around the aggregate because $Ca(OH)_2$ (or CH) tends to deposit at the surface of the aggregate during hydration.

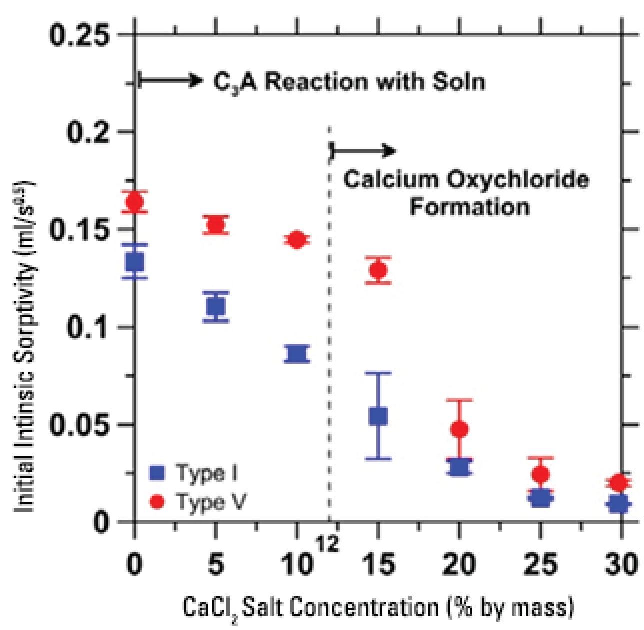


Figure 2-6. Reduction in the rate of water absorption due to salt reactions

Acoustic emission (AE) was used by Farnam et al. (2015a) to listen for crack development in samples saturated with 0, 5, 15, and 29.8 percent $CaCl_2$ solutions (Figure 2-7). For the lower concentrations, a small amount of cracking was noticed as the temperature decreased from room temperature. These cracks are due to the difference in the coefficient of thermal expansion between the aggregate and the matrix. When the freezing begins to occur (-5°C for 0 percent $CaCl_2$, -8°C for 5 percent $CaCl_2$, and -12°C for 15 percent $CaCl_2$), however, the acoustic activity increases dramatically, indicating that cracking is taking place because of the expansion caused by the ice formation inside specimen pores (Figure 2-7c). Cracking, however, begins at higher temperatures for samples with a higher concentration upon contact with solution for specimens saturated by 20, 25, and 29.8 percent $CaCl_2$ (shown in Figure 2-7d). It should be noted that the samples in Figure 2-7 were stored at 23°C (which is consistent with calcium oxychloride; see Figure 2-5), because the formation of calcium oxychloride can cause cracking due to volume expansion.

Note that in advanced stages of deterioration, both saturated-related freeze-thaw damage and damage due to salt attack may likely occur, resulting in large areas of material loss and spalling around the joints.

Potential mitigation methods

Although the chemical reactions that occur between deicing salts and the cement matrix can be quite complex, simple steps can be taken to reduce the potential for salt-related damage to occur (Suraneni et al. 2016a):

- Use supplementary cementitious materials (SCMs) in the mixture.

  This reduces $Ca(OH)_2$ (or CH) in concrete due to both dilution and pozzolanic reactions (Weiss and Farnam 2015), which in turn reduces the formation of calcium oxychloride. Low temperature differential calorimetry offers a

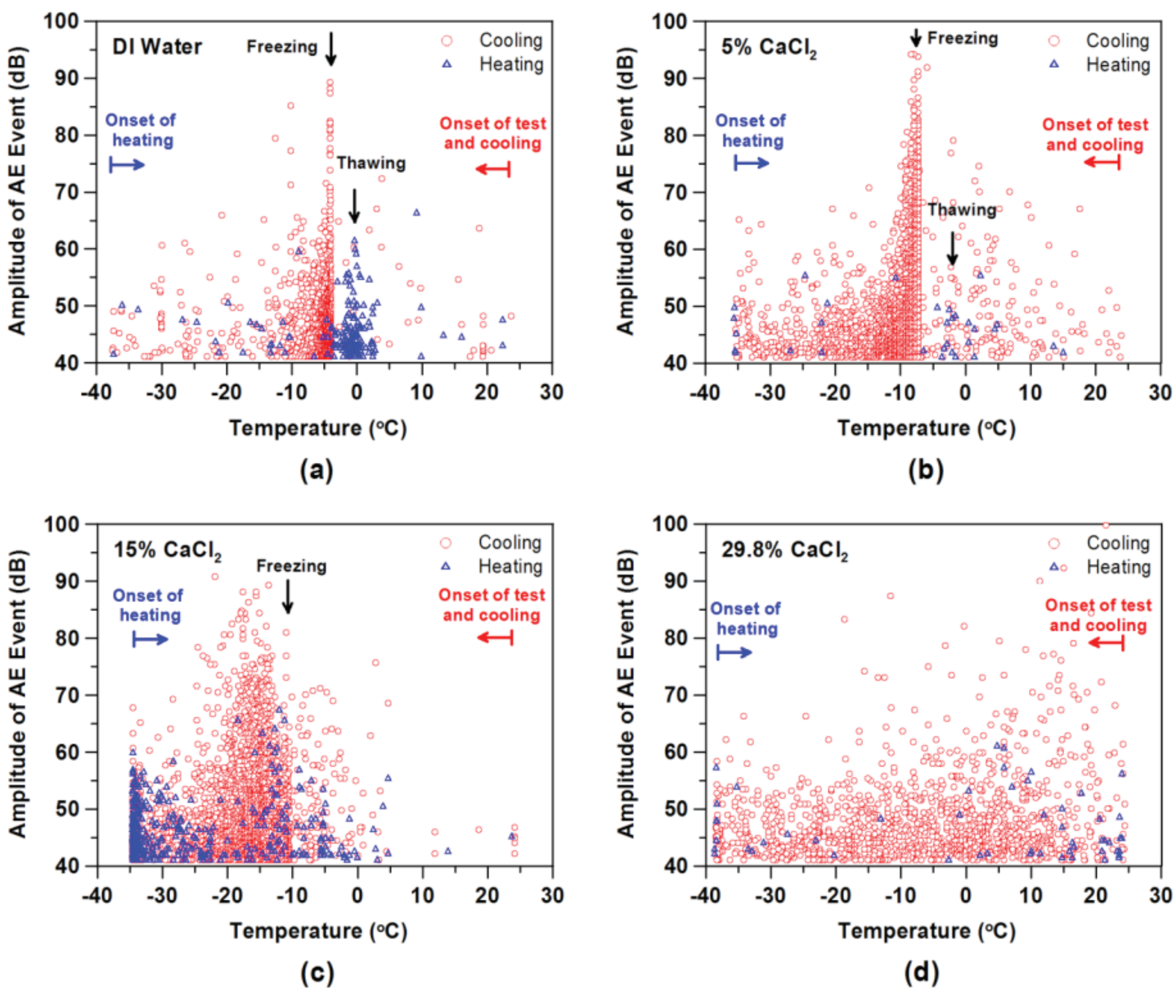


Figure 2-7. Passive AE events versus temperature during cooling and heating for mortar specimens saturated with (a) deionized water; (b) 5 percent $CaCl_2$ solution; (c) 15 percent $CaCl_2$ solution; and (d) 29.8 percent $CaCl_2$ solution (scattered AE activity can be seen for the 29.8 percent concentration)

method to quantify the amount of calcium oxychloride that may be produced by a given binder (Monical et al. 2016 [in press]). (Note that while increased SCM replacement works well for jointed plain concrete pavements and unreinforced sidewalks, the reduction in the pH of the concrete matrix may impact corrosion of reinforcing steel in reinforced concrete.)

- Use concrete mixture designs that provide low w/c ratios to reduce concrete fluid transport.
- Have entrained air-void systems with appropriate parameters for frost resistance.
- Emphasize proper testing of constituent materials.

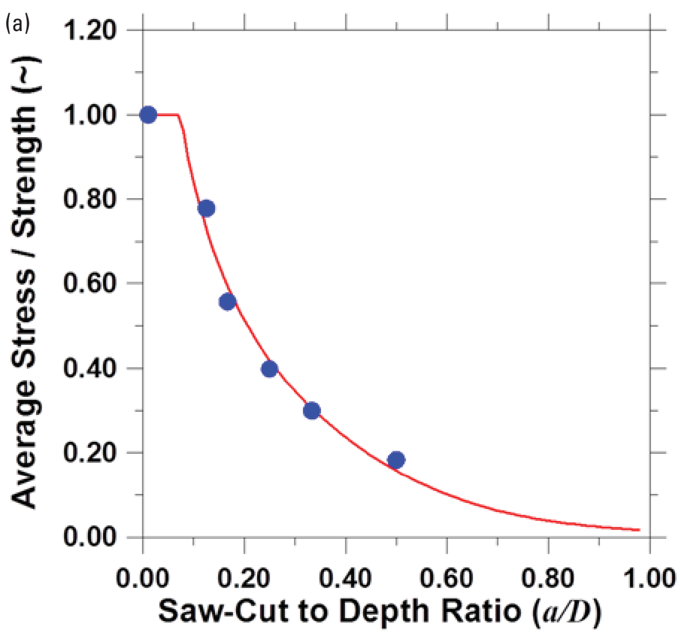
Mechanical Damage

In many pavement texts, joint sealants are promoted as a way to keep debris out of the pavement joints, thereby promoting long-term durability (Yoder and Witzak 1975). Taylor et al. (2011) discussed that, while joint damage can occur due to debris trapped in joints, they did not think that debris-related damage was significant in concrete pavements experiencing the type of premature joint damage discussed in this report. This may be because, at interstate speeds, traffic can lift debris from joints. (Note that debris in joints may be related to joint damage later in pavement life.)

Taylor et al. (2011) reported that spalling can occur at the joints of concrete pavements when they are sawed too early and that damage can occur at the bottom of the saw cut when sawing machines are used with worn bearings or an inappropriate blade or when cutting on a curve. Thier (2005) conducted studies of joint construction, and Raoufi et al. (2008) simulated stress development at the saw-cut joints. It was reported that saw cutting needs to be performed before the stress reaches the tensile strength of the pavement to reduce the potential for random cracking (Raoufi et al. 2007), and this stress reduction factor is related to the depth of the saw cut being used. Figure 2-8 shows that deeper saw cuts require a greater reduction in the stress-to-strength ratio used for saw cutting. It was also observed that micro-cracking can occur at the base of the saw cut when the stress in the pavement is high (Castro et al. 2011), increasing the rate of saturation and freeze-thaw damage (Yang et al. 2004; Weiss 2016).

It has been reported that mechanical loading may play a role in damage at joints due to curling (Monical et al. 2016a) or shear from traffic, especially at early ages. The reason that such damage is more frequently related to locations such as ramps and intersections may be that the contractor has switched to straight cement mixtures (i.e., mixtures that do not contain SCMs) at those locations to allow earlier opening of the pavement to traffic. If straight cement mixtures are used in these locations, they may result in a greater amount of calcium hydroxide (CH) formation which, when combined with potentially greater salt application rates, can result in the potential for calcium oxychloride formation.

Chapters 3, 4, and 5 summarize best design, construction, maintenance, and preservation practices that result in long life concrete pavements with durable joints.



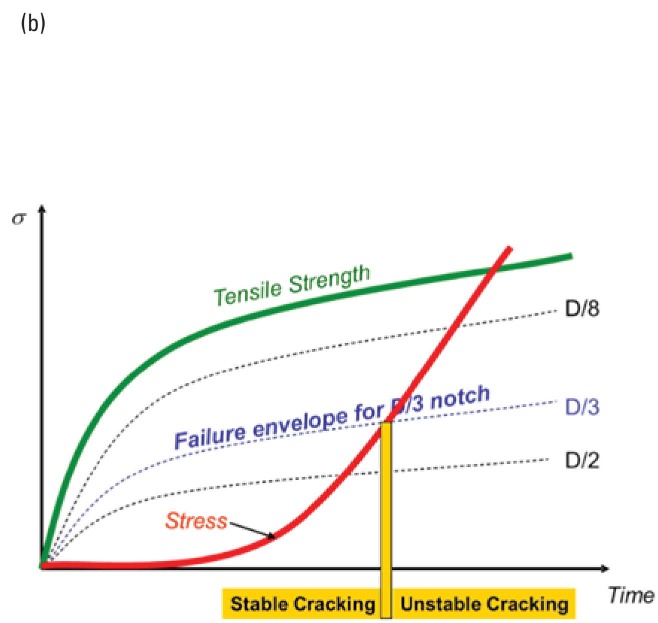


Figure 2-8. (a) Strength and stress development for concrete pavements as a function of saw-cut depth, and (b) the saw cut stress-to-strength ratio

CHAPTER 3. PREVENTING JOINT DETERIORATION IN NEW PAVEMENTS

To construct a new concrete pavement with durable joints that are resistant to premature deterioration, it is important to use a holistic approach that optimizes all aspects of the pavement system. Using a concrete mixture that is resistant to freeze-thaw damage and salts-related damage, as discussed in the previous chapter, is just one element of such an approach. It is also critically important to design and construct a pavement that will be relatively dry. This is accomplished by taking the necessary measures to ensure that groundwater will be drained away from the pavement structure and to minimize the ingress of precipitation and meltwater into the concrete structure. Finally, a critical element of a durable pavement system is the supporting layers and their strength, stiffness, drainage, and resistance to weathering.

Furthermore, every design and construction decision must be balanced against the availability and cost of materials, construction speed, local conditions, and construction schedule. This means that the optimal solution in one location may not even be considered in another.

Soil, Subgrade, and Base Systems

When deterioration occurs within a concrete pavement, it is common for engineers to focus their attention on the concrete itself and not think about the impact of the supporting layers. The performance of a pavement, however, is strongly influenced by the underlying materials. These materials are responsible for providing uniform bearing strength, resisting volume changes over time, and removing moisture from the pavement system.

The proper removal of water from the concrete pavement joint system is an important aspect of preventing joint spalling, particularly in cold-weather states. Poor drainage can decrease the drainage rate of a pavement and can cause water to stay in a joint, exacerbating joint deterioration. Furthermore, poor drainage can cause loss of support and stiffness of the base layers. This will quickly lead to failure of the pavement from surface loading. A drainage layer should be considered in the initial design phase when high water tables or excessive moisture is anticipated during the expected life of the pavement system.

There are many possible base systems. The following section provides a general overview and describes the importance of each. Note that it is possible to provide a long-lasting pavement with any of the following base systems as long as the pavement is designed and constructed correctly.

Natural subgrade

The natural subgrade is the native soil at the site. Subgrades can be classified based on the proportion of coarse- and fine-grained fraction, plasticity, or the response of the fine-grained fraction to moisture. The classification of soils can be done by the American Association of State Highway and Transportation Officials (AASHTO) or American Society for Testing and Materials/Unified Soil Classification System (ASTM/USCS).

Based on the characteristics of the layer, the modulus of subgrade reaction, k, can be determined. This is an estimate of the support the subgrade will provide. Because of the ability of a concrete pavement to spread loads over a wide region, most pavement design methods are not sensitive to the value of k. The water table level and the subgrade's susceptibility to moisture and frost have a much greater effect on pavement performance.

In some cases, the natural subgrade for a roadway may be enough to carry the design load for the design life. Because of the variability of soil, however, it is rare to find a natural subgrade that provides a soil with uniform properties. Because of this, it is necessary either to remove undesirable material (organic deposits, silt, and sometimes clays) or to stabilize the soil with additives.

Stabilized subgrade

Stabilization of soils has become very common in highway construction. To stabilize the soil, lime, fly ash, cement kiln dust, or portland cement is added to an existing subgrade to improve low strength soil, reduce moisture swelling, and improve construction methods. In fact, soil stabilization can greatly reduce weather-related construction delays and accidents and improve contractor productivity by providing a stable work platform. The design of stabilized subgrades is outside the scope of this document, but the Portland Cement Association (PCA) and American Concrete Pavement Association (ACPA) provide an excellent document on the subject (PCA/ACPA 1991), as does the Innovative Pavement Research Foundation (Hall et al. 2005). In general, lime, some fly ash, and cement kiln dust can be used to treat expansive soils such as clays. Fly ash and portland cement is more often used to stabilize soils with high silt content or coarse-grained soils.

Granular subbase

The use of granular subbase for concrete pavement has many benefits, including a stable working platform that can reduce construction delays. These materials provide a high subgrade stiffness and strength value. If designed correctly, the drainage of these layers is outstanding; it has been reported these subbase systems can produce the smoothest pavements. Most high-volume pavements and airport runways use this type of pavement system. Unfortunately, this subbase is the most expensive. If the subbase is expected to also function as a drainage layer, however, then this is the best choice.

Thicker base layers are less likely to clog and should be considered where long-term performance is needed or in regions with water tables near the surface. The increase in thickness of drainable layer may not increase the rate of drainage, but it should still increase the life span of the layer.

Drainage of pavement system

The main objective of any subsurface drainage system is to remove excess water from the pavement structure, then deflect the water to an appropriate location. Excessive water can cause severe damage to the subbase and also the concrete. Joint deterioration related to saturated concrete, discussed in chapter 2, requires water sitting in the joint for extended periods. Removing water from the joints mitigates these failures. In addition, standing water on a pavement increases the risk of hydroplaning and decreases visibility. The water can come from the surface from precipitation or from groundwater flowing beneath the pavement.

As discussed previously, all portions of the pavement must be designed to work together to produce the proper performance. Figure 3-1 displays a typical profile for a concrete pavement with a basic drainage system.

The surface of the pavement, shoulders, and surrounding right of way should be sloped to allow water to be carried away. Even with well-designed surface drainage, however, water will always enter the pavement structure through cracks and joints. If this water is not removed from the pavement, it will cause long-term damage.

One way to remove this water is to provide a drainage system or drainable layers within the subbase. These layers generally have a higher cost and can sometimes be less stable (because of their high porosity); however, any decrease in support stiffness is a trade-off in long-term performance. In fact, unusual joint damage has been observed in pavements in which undrainable bases are used (Burnham and Rohne 2011) and the joints retain fluid (Weiss 2007). Drainable layers are typically between 4 in. and 6 in. thick and often use an open-graded aggregate such as a #57 or #67 with no sand. These layers can remove water quickly, with values as high as 150 ft/day suggested for highway pavements.

It is common to ignore drainable layers in the design of the pavement because of their low stiffness and the difficulty in predicting their properties. If there is concern about the performance of these layers, they can be stabilized with portland cement. There is not a lot of guidance on how much cement to use in the mixture. The ACPA suggests at least 250 lb/yd^3. This stabilization will also greatly help these layers with construction traffic.

Because of the open-graded nature of a drainable layer, care should be taken not to allow cement paste from the concrete pavement slab to penetrate this layer and cause a bond to form between the slab and the drainable layer. Also, the other subgrade layers may contain fines that can enter the drainable layer and clog it. In this case, the use of a geotextile between the drainable layer and surrounding layers is recommended. To be effective, the opening size in the selected geotextile must be smaller than the size of the fine materials.

Once water enters the permeable base, it has to exit the pavement system. This is accomplished by installing longitudinal drains or by extending the drainable layer until it reaches the side slope of the pavement, which is called “daylighting.” Daylighting is economical but leaves the drainable layers more susceptible to clogging and the need for future maintenance. Therefore, the use of longitudinal pipes or French drains has proven to be more reliable. The capacity of the drain must be checked and an adequate slope must be used to move the water to the outlets. It is common to use pipe sizes of 3 in. to 4 in. with an outlet spacing of roughly 250 ft. Geotextile fabric is also commonly used with edge drains to keep them from clogging.

The Federal Highway Administration (FHWA) has produced a useful tool, DRIP 2.0, to help engineers design their drainable layers; see www.me-design.com/MEDesign/DRIP.html. A user guide by Mallela et al. (2002) is one of several publications with specific design examples.

Concrete Mixture Recommendations

Regarding materials selection and mixture design, the key variables are w/c ratio, use of SCMs, and proper air entrainment. As discussed in chapter 2, the hardened cement paste is susceptible to both physical and chemical attack mechanisms. The impact of both attack mechanisms is most detectable at the joint where the potential for exposure and entrapped water is the highest. The following factors should be considered when exposure to freeze-thaw cycles and deicers occurs.

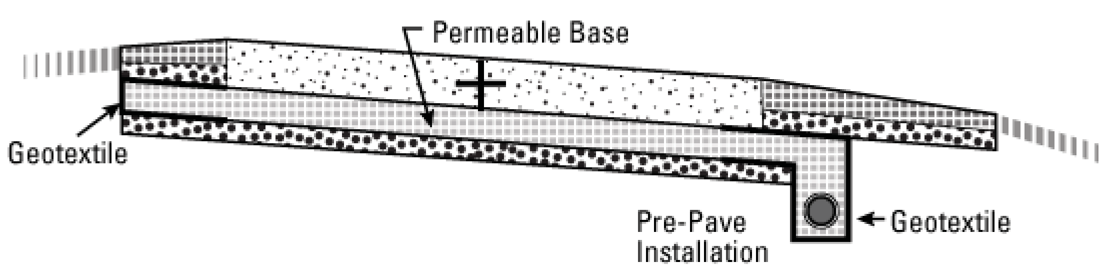


Figure 3-1. Typical cross section of subsurface drainage system

Performance of Iowa Pavements (TR-640)

A recent study in Iowa (White and Vennapusa 2014) found that a wide range of pavement conditions and foundation layer support values exist despite a narrow range of values used for design. The calculated design inputs for the base layers are often different than typically is assumed. This was true for all of the materials tested. Because of this, a recommendation was made to incorporate field testing to verify the selected pavement design values used.

In this study, 16 different pavements on city and county roads in Iowa were tested. The sites were in service between 30 days and 42 years and had a variety of surface conditions (from poor to excellent). These pavements also had varied support conditions, traffic, and pavement thicknesses. At each site, a visual pavement condition was determined. A falling weight deflectometer was used to determine the support layer stiffness and load transfer efficiency at the joints. A dynamic cone penetrometer was used to study the base-layer conditions. In addition, the support layer drainage was inspected by using a newly developed technique called the core hole permeameter.

The study found that improving subgrade strength/stiffness within the top 16 in. of the subgrade layer, improving drainage, and reducing variability led to improvement in pavement performance for Iowa pavements. Also, subgrade layer properties can be improved by stabilization, and drainage can be improved with only a thin drainage layer (as little as 3 in.). An important finding from the study was that the reliability of pavement properties and performance could be improved by using in-place testing. It was recommended that in-place testing become an important part of pavement projects in the future.

Water-cement ratio

Maintaining a relatively low w/c ratio is the first approach to mitigating joint deterioration via mixture design. As the w/c ratio is reduced, the concrete will have lower paste permeability and higher strength. For freeze-thaw durable concrete in severe exposure, ACI (American Concrete Institute) 201 recommends a w/c ratio of 0.45 maximum (ACI 2008). For reduced risk, it is recommended that designers consider a w/c ratio of 0.40 to help ensure a durable cement paste.

Permeability

When cement and water are first mixed, there is a dispersion of cement grains as shown in Figure 3-2(a). The cement and water chemically react and hydration products form as shown in Figure 3-2(b). The initial hydration products block the water from access to the cement, and the hydration process slows. For further hydration, the water must diffuse through the early hydration products, which is a slow process. To continue hydration, water must be maintained in the pore structure. (This latter step is also central to curing, which will be discussed later in this chapter.) As hydration continues, the hydration products fill the water-filled space between cement grains. Hydration continues until the water can no longer reach the cement grains, the water is consumed, or the water is lost to evaporation. As shown in Figure 3-2(c), the remaining void space surrounding the cement grain is referred to as capillary porosity; the volume of this space is determined largely by the w/c ratio of the original mixture. Entrained air voids also exist within the hydration products and, together with the capillary voids, form a pore network, as shown in Figure 3-2(d). Capillary porosity is a key component of the overall concrete porosity, and controlling it through control of the w/c ratio is a necessary step for controlling water ingress into the concrete.

Strength

As described in chapter 2, the expansive force caused by water freezing within concrete is one of the most destructive forces concrete will experience. As a result of water expanding when it transforms from liquid to solid, the concrete is internally placed in tension, the scenario in which concrete has its lowest strength. Ensuring sufficient strength will help produce durable concrete for freeze-thaw exposure. The strength of concrete relates directly to the hydration process discussed previously. Strength increases as the cement more completely hydrates. This results from the reduction of capillary pore space and the production of more cementing phase (i.e., calcium hydroxide and calcium silicate hydrate, or CH and C-S-H in cement chemistry shorthand).

It has been known since the work of Abrams (1918) that concrete strength is inversely proportional to the w/c ratio of the concrete mixture (Kosmatka and Wilson 2016). For all these reasons, controlling the w/c ratio for the purpose of controlling strength is critical.

Supplementary cementitious materials

Supplementary cementitious materials improve concrete's freeze-thaw durability by (a) consuming CH and thereby minimizing the negative aspects of CH in concrete, and (b) increasing the volume of cementing phases within the hardened cement paste and consuming void space (i.e., capillary void space), reducing concrete permeability.

The hydration of cement essentially produces two principal phases, CSH and CH. The C-S-H phase is considered to be the "cementing" phase whereas the CH phase is present but has little cementing value. The CH forms in platelike crystals that easily fracture. More important, CH is soluble in water. Further, the solubility of CH increases as the temperature decreases. This means when concrete is exposed to water at a temperature near the freezing point, the CH will dissolve more readily. Some of the dissolved CH may re-precipitate within the concrete, but some will exude or otherwise be lost. This process creates an increased permeability and more water ingress, and the process accelerates. As described in chapter 1, the CH is a reactant in the calcium oxychloride

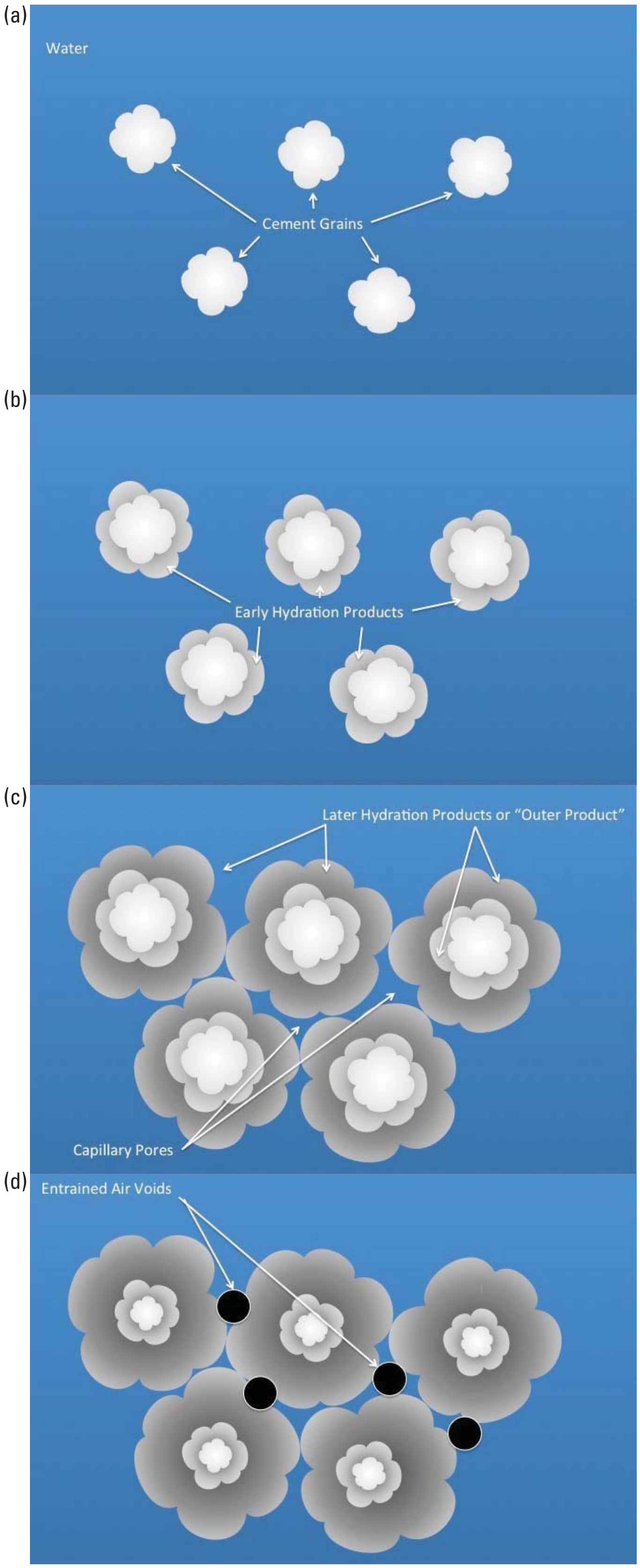


Figure 3-2. Cement grain dispersion and development of hydration products and capillary pores

attack mechanism, meaning CH dissolution in the presence of deicing salts is a key mechanism in the calcium oxychloride attack mechanism.

All SCMs provide a filler effect in which concrete permeability is reduced by unreacted SCM particles or SCM reaction products occupying void space that would normally exist within the C-S-H phases. Because of the desire to consume CH, pozzolanic materials are the first choice for SCMs, and these include Class F fly ash and silica fume. Research has clearly shown, however, that slag cement (Sutter 2008) and Class C fly ash will also improve concrete freeze-thaw durability in the presence of deicers, although larger replacement rates are typically required (e.g., 35 to 60 percent). Agencies should consider cement replacement rates of 20–25 percent Class F fly ash or 30–35 percent Class C fly ash or a combination of 20 percent slag and 20 percent Class C fly ash.

Well-graded aggregates

Aggregate with the largest nominal maximum size that is practical for job conditions should be used (Taylor et al. 2006; Kosmatka and Wilson 2016). A larger maximum aggregate size reduces the required cement paste volume, which helps control shrinkage and also reduces the amount of paste available to undergo chemical and physical distress mechanisms. For pavements, both ACI and PCA recommend the maximum aggregate size should be less than one-third the slab thickness and three-quarters of the free space between reinforcing bars or reinforcing bars and formwork (ACI 1991; Kosmatka and Wilson 2016).

An even greater reduction in paste volume can be achieved by using a well-graded, or "optimized," gradation. An optimized gradation uses an intermediate aggregate size (e.g., ~0.33 in.) that fills the void space between the coarse aggregate particles. By reducing the cement paste content, all previously discussed paste-related durability issues are reduced. Therefore, well-graded aggregates will improve concrete freeze-thaw durability.

Air-void system

As discussed in the previous chapter, it is well known that proper air entrainment is critical for freeze-thaw durability in concrete. To protect concrete under these conditions, it is necessary to achieve an adequate air-void system. For concrete with an aggregate top size of 1 in., ACI 201 recommends an air content of 6 percent for severe exposure. Most DOT specs fall within the range of 5 to 8 percent, with 5 percent minimum in place. It is important to measure the air content after the paver, or in the hardened concrete, to ensure an adequate air-void system.

There are many theories regarding how air-void systems function. Powers (1945) attributed freeze-thaw damage to excessive hydraulic pressures resulting from the expansion of ice. Regardless of the theory applied, void spacing is critical to proper functioning of the air-void system and, in turn, attaining the proper air-void system is determined by the correct air entrainment admixture (AEA) dosage. The dosage of a given AEA required to yield a suitable air-void system is typically well established by manufacturers, but it must be confirmed by performing laboratory-scale mixture design tests, especially when the use of SCMs such as fly ash are contemplated.

As described in the previous chapter, recent work by Todak et al. (2015) has centered on the significance of "critical saturation" in joint deterioration. Building on the work of Fagerlund (1977), Li et al. (2011) has shown that once the void space in concrete becomes critically saturated, meaning approximately 85 to 88 percent of the void space is saturated, concrete will crack with only one freeze-thaw cycle. Air entrainment provides more void space, prolonging the time to critical saturation. For joint deterioration, especially where joint sealants have failed or joints may not be draining, super-cooled brines can accumulate in joints and the concrete nearest the joint could become critically saturated whereas the remainder of the slab is not. Then, with only a few freeze-thaw cycles, cracks can initiate. Additionally, salts hold moisture in the concrete and can work to increase the saturation of concrete (Spragg et al. 2011).

Construction Practices

Construction practices that impact joint deterioration include curing, sawing, and sealing.

Curing

As discussed to this point, the properties of the hardened cement paste strongly influence freeze-thaw durability. Without question, many properties of the hardened cement paste are improved through proper curing. The purpose of curing is to promote hydration by maintaining the necessary moisture and proper temperature of the concrete immediately after placement and finishing (Taylor 2014; Kosmatka and Wilson 2016). Inadequate curing can result in surface damage in the form of plastic shrinkage cracking, scaling, spalling, and weakening of the paste.

Historically, wet curing (e.g., ponding or fogging, wet burlap, and plastic sheeting) has been used and offers the best performance; however, for paving projects it is not practical. The application of liquid membrane-forming curing compound is the most commonly used method of curing. These are organic coatings placed over the surface of the concrete to reduce the rate of pore water loss (Taylor et al. 2006). Curing compounds are temporary and will break down and degrade with exposure to sunlight and traffic.

General guidelines on the use of curing compounds include the following (Van Dam et al. 2002; Kohn et al. 2003; Poole 2005, 2006; Taylor et al. 2006, Hajibabaee et al., 2016):

- Do not apply curing compound to concrete that is still bleeding or has a visible sheen of water on the surface.
- Apply the curing compound immediately after texturing. Any delay, particularly during hot, windy conditions, can cause significant harm to the concrete resulting in plastic shrinkage cracking.
- Apply the curing compound uniformly to the concrete surface, ensuring that both the top of the slab and the sides are adequately covered. (Applying two thin coats is preferable to applying one thick coat.) For fixed-form paving, the vertical edges should be coated after form removal. Automated application equipment is more effective at providing uniform coverage, and hand-operated equipment should be used only on small areas.

Sawing joints

Although sawing has been suggested as a potential cause of premature joint deterioration in concrete pavements, there has been limited research regarding what if any impact sawing may have on joint deterioration. Clearly, if done improperly, the resulting joint will be bruised, raveled, or marred by micro-cracks emanating from the sawn joint. These defects will contribute to joint distress. But the question still remains whether or not sawing, when done properly, causes weakening of the concrete that may lead to joint deterioration.

Adhering to sound sawing practices is advised to minimize the risk of joint deterioration. There are numerous considerations when selecting either a conventional or early-entry saw. The blade must be in good condition but, more important, it must be the correct blade for the type of materials being cut. If in doubt, the saw blade manufacturer should be consulted to ensure the correct blade is selected. Also, the saw needs to be operated at the proper operating speed, again established by the manufacturer. Operating at the wrong speed can lead to blade wobble and excessive micro-cracking in the concrete.

Most important, select the correct sawing window. There is no one correct answer for when to saw. The window is determined by the mixture and conditions of placement. In general, the sawing window begins when concrete strength is sufficient for sawing without excessive raveling along the cut; see Figure 3-3. Taylor et al. (2006) propose that the sawing window ends when random cracking starts to occur. Raoufi et al. (2007) have suggested that the sawing window ends as cracking transitions from stable to unstable at the saw cut,

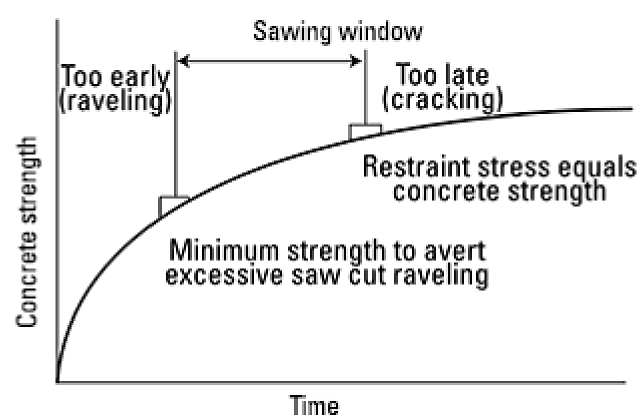


Figure 3-3. Sawing window

as shown in Figure 2-8 in the previous chapter. Taylor and Wang (2015) discuss a device (P-wave propagation) that can predict the sawing window for early-entry and conventional saws.

It is challenging to inspect how well joints are constructed and how they are performing over time (Harris et al. 2015). It is important, however, to ensure that saw-cut joints activate. If the concrete does not crack through at the base of the cut, the saw cut acts as a reservoir that holds water and brine and creates saturation of the concrete (Raoufi et al. 2007). If it is discovered that saw-cut joints are not activating, adjustments should be made to the cutting depth or other changes considered to correct the problem.

Sealing joints

There is debate on whether or not joint sealant is needed (Ray 1980). Many states still use a widened saw cut and joint sealant. If sealant is used, it must be maintained; see chapter 4. In many concrete pavements, joint sealants have failed, resulting in the ingress of water. To minimize the potential for joint sealant failure, care must be taken during installation (ACPA 1995; Lynch et al. n.d.).

While joint sealants are helpful, they typically need to be replaced. Therefore, it is recommended to use a drainable base that allows water to leave the joints. This early investment in the drainage of the pavement will help to greatly prolong the life of the pavement.

Surface sealers

Penetrating sealers or surface sealers are gaining attention as a means to reduce moisture ingress into concrete. Penetrating sealers may significantly impact the ingress of water into concrete and can be an excellent maintenance option for protecting concrete in a wet-freeze environment. Research into the use of surface sealers is ongoing; the following paragraphs summarize general conclusions to date.

It has been proposed that instead of or in addition to installing joint sealant, coating joints with a surface sealer may reduce joint damage caused by fluid ingress or salt reactions (Weiss et al. 2007). Surface sealers are easy to apply on the horizontal pavement surface near the joint. Getting the sealer to penetrate and adequately coat the vertical surfaces of a joint is more difficult. In most cases, it is necessary to saturate the concrete with the surface sealer to allow for penetration; this is problematic within the joint.

Surface sealers have been shown to have varying levels of success in reducing joint damage in concrete pavements. Although some such treatments have shown advantages in preventing salt ingress, others appear to provide a physical barrier between the CH and the salt (Wiese et al. 2015) that can reduce or prevent calcium oxychloride formation. Some surface sealers (film-forming treatments) have been shown to be permeable to fluids during temperature changes, which would not have a very beneficial impact on calcium oxychloride formation (Coates et al. 2009; Harris et al. 2015).

There are many types of surface sealers and many variables that affect their performance. Two common sealers are based on either silane or siloxanes. These sealers are in the general class called “hydrophobes,” which means they make the concrete surface hydrophobic (i.e., water hating). They keep liquid water from wetting the surface and penetrating the concrete while allowing water vapor to escape from the concrete. These materials can be formulated as either water-based or solvent-based mixtures. Either formulation can perform well; the choice must be made in association with other factors, including volatile organic compounds (VOC) content and solids content.

A VOC is an organic (i.e., carbon-based) compound that can combine with other gases in the atmosphere, in the presence of sunlight, to form ozone and contribute to smog. In general, water-based formulations are lower in VOC content compared to solvent-based surface sealers. This is not universally true, however; not only is the solvent volatile but also the active component in the sealer (e.g., silane, siloxane) is volatile. Always confirm the VOC content by consulting the product safety data sheet.

Confirming the allowable VOC limit for a surface sealer used in a specific project is less straightforward. The Environmental Protection Agency has set limits on the VOC content of sealers. The 1999 Architectural Coating Rule for Volatile Organic Compounds (63 FR 48848) set limits that vary with geographic location; the rule also established 61 different subcategories to classify sealer products, and the VOC limits vary by subcategory. Further complicating matters, state regulations supersede federal regulations. Therefore, consult state environmental agencies to confirm the allowable VOC content.

The solids content of a surface sealer is the content of active component of the formulation (e.g., silane, siloxane) with values typically ranging from 10 to 40 percent weight.

Use and limitations of surface sealers

One of the first lines of defense against premature joint deterioration is to design concrete pavement mixtures with the desired performance in mind, following the recommendations in this document. However, many factors (e.g., material availability, cost) affect pavement design and, therefore, a second line of defense may be the application of surface sealers on the joints of new pavements.

Note that surface sealers will not remediate concrete that has already failed and can provide only marginal protection for concrete that is not properly designed and constructed with quality materials or that is not properly placed and cured.

Generally, sealers having higher solids content perform better compared to sealers with lower solids content, but, again, this trend is not universal. A side effect of an increased solids content is a decrease in the solvent content and with it a potential decrease in VOC content.

Another type of surface sealer is based on soy methyl ester (SME). Golias (2010) and Golias et al. (2012) compared SME-polystyrene sealants (SME-PS) with two silane sealants. It was observed that concrete samples treated with SME-PS did not show damage. The silane sealants appeared to be vulnerable to thermal contraction, which caused the sealant to crack or become permeable. On the other hand, SME-PS absorbed into the pores and remained flexible during freezing. Additional research is needed to better understand the role of topical treatments.

Field trials using SME-PS were initiated in 2011 in central Indiana on US 231, which was built in 1999, and on 126th Street in Fishers, Indiana. These trials have been documented (Wiese et al. 2015), and the joint condition was video-recorded for later reference so that any areas of damage can be monitored over time. Core samples taken after three years showed little damage due to calcium oxychloride; however, the cores revealed inconsistencies in the joint depth achieved during sawing. Further research is needed to provide long-term documentation of field tests (Jones et al. 2013; Thomas 2016) and to provide guidance on application rates and timing.

CHAPTER 4. MAINTENANCE ACTIVITIES TO REDUCE JOINT DETERIORATION RISK

While our infrastructure is designed to require as little maintenance as possible, there are several critical areas in which maintenance activities must be performed to reduce risk of concrete pavement joint deterioration. This includes routine maintenance to the pavement system, as well as winter maintenance to ensure that the surface is free of snow and ice.

Routine Maintenance

Routine maintenance to prevent or mitigate premature joint deterioration includes joint cleaning and sealing, application of surface sealers, and maintaining surface and subsurface drainage and drainage systems.

Joint cleaning and sealing

A substantial portion—as much as 65 to 80 percent—of water that enters a pavement from the surface does so through the longitudinal joints or cracks, typically through the lane/shoulder joint. If joint sealants fail, water can directly enter the joint. If the base layers are not designed to drain water away, then the joints will remain full of water; this has been a major contributor to joint failure in concrete pavements. Also, if the sealant fails locally only and remains in the joint, then evaporation within the joint is reduced (Weiss et al. 2011). The longer water sits in the joint, the more it will penetrate the concrete and cause deterioration. For these reasons, special attention needs to be given to maintaining joint sealants.

Based on a review of a number of available studies, the performance of concrete joint resealing ranges from two to eight years, while the performance of concrete crack sealing ranges from four to seven years (Peshkin et al. 2011), with failure defined as 25 percent of the sealant installation being no longer functional. Longer performance lives are possible, however. For example, using nearly seven years of performance data, the SHRP H-106 joint resealing experiment extrapolated the performance life of several silicone sealants to be between 12 and 16 years (Evans et al. 1999).

Joints should be inspected every three years. It is critical to ensure that joints are free of incompressible materials and that the joint sealant is performing properly. Joint cleaning can be daunting because of the large number of joints in a pavement; however, if joints are not maintained, they will fill with incompressible materials. These materials will keep building until damage is caused in the pavement.

On a multilane pavement, the lanes can be closed in phases and the joints can be cleaned with high-pressure water. Next, the joint sealant should be visually inspected for local areas of debonding or failure. This can be done by first placing plumber's putty or modeling clay near the end of the joint in the closed lane. The joint can then be filled with water and inspected for bubbles at the regions between the sealant and the concrete. These bubbles are caused by water entering around the sealer and water escaping. Also, the water level should not be changing. Although this method works, it can be daunting to complete on miles of pavement.

When the joint sealant is severely damaged, all of the sealant should be removed from that joint. After removal, the joint should be inspected to see if water is standing in the joint. If water is present, it should be removed. This can be accomplished with either clean compressed air or a vacuum. After the water has been removed, the joint should be resealed.

Surface sealers

Routine maintenance of existing pavements may include the application of surface sealers to the joints, instead of or in addition to installing joint sealant, to reduce potential damage from moisture ingress into the concrete or salt reactions. It is important to thoroughly coat the vertical surfaces or faces of the joint with the sealer.

Surface sealers must be re-applied periodically. The re-application cycle depends on the sealer, the exposure, and the wear; research is ongoing. For example, Moradllo et al. (2016) found that, based on the performance of 60 bridge decks, no silane-based surface sealer failed before 12 years. By 18 years, however, close to 50 percent of the sealers had failed.

Surface drainage

Surface drainage is relatively easy to maintain. The side ditches of the pavement should be mowed and cleared of debris, and buildup of soil on the pavement surface should be removed. Localized surface grinding of the pavement can also be used to restore the correct profile or, in some cases, prevent isolated ponding of water.

Subsurface drainage

Drainage layers and edge drains must be inspected to determine if they continue to allow water to leave the pavement. If longitudinal drains have been used, the pipes can be cleaned. If the drainage layer is daylighted or tied to an existing storm sewer system, it should also be inspected for fines buildup. Site visits after a rain event should show evident of use or activity if they system is draining properly. Where poor drainage is suspected, the pavement edges can be exposed and the clogged regions of the drainable layers removed and replaced with clean materials.

Maintaining subdrain systems or installing retrofit drainage systems

A recent study in Iowa found that about 65 percent of drainage outlets on concrete pavements are blocked (Ceylan et al. 2013). If the pavement system has an existing drainage system, such as subdrains, it is critical that the drainage inlets and outlets be cleaned and inspected periodically, approximately every two years.

Restoring drainage to an existing pavement with no drainage layer can be challenging. When an existing pavement begins showing signs of spalling due to moisture, one retrofit strategy is to install edge drains along an existing pavement. The edge drains can lower the local water table.

As shown in Figure 4-1, the drainage path to an outlet can be shortened, particularly when an existing subdrained system is plugged or there is an impervious earth blockage in the shoulder area. The shortened drainage path is accomplished by connecting an aggregate outlet ("French" drain) into the existing granular subbase and daylighting it out to the shoulder foreslope area, ditch, or existing storm sewer structure or installing a pipe edge drain and outletting it to a low point in the profile grade to the foreslope or ditch.

Nationally, the performance of pavements with retrofitted edge drains has been mixed. Overall, inconsistent performance of retrofit edge drains has been mostly attributed to a combination of improper usage, improper design, damage during installation, lack of post-installation maintenance, or failure to provide other pavement repairs that are needed when the edge drains are retrofitted. For proper selection and guidance for the installation of retrofitted edge drains, refer to the *Concrete Pavement Preservation Guide* (second edition) (Smith and Harrington 2014).

Winter Maintenance

Winter maintenance-related issues include managing deicing/anti-icing chemicals and allowing an adequate maturity period for new pavement.

Deicing/anti-icing chemicals

Deicing and anti-icing chemicals are normally brines, which are a mixture of water and a dissolved salt. The various salt brines used in roadway winter maintenance can have different effects on the durability of the concrete, depending on the salt used and its concentration in the brine solution. As mentioned earlier, sodium chloride (NaCl) brines may be recommended, rather than magnesium chloride ($MgC1_2$) and calcium chloride ($CaC1_2$) brines, because sodium chloride is not as reactive with cement paste.

Salt brines function as deicing and anti-icing chemicals by reducing the freezing temperature of water. Table 4-1 shows the maximum freezing points for four common salt brines; in practice, the temperatures at which these products lose effectiveness is higher than the temperatures in the table.

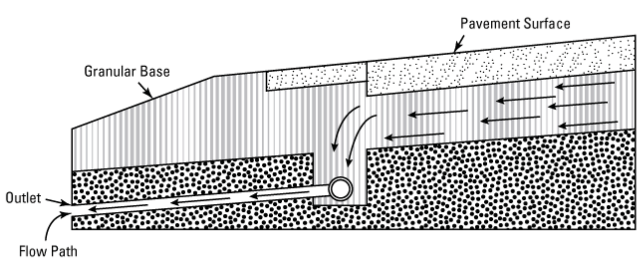


Figure 4-1. Example of shortening drainage path to an outlet

Figure 4-2 shows the partial phase diagrams for mixtures of water and common deicing/anti-icing salts. The phase diagrams indicate the freezing point of a brine solution as a function of the concentration of the chemical. Note that for all brines shown, a minimum freezing temperature is obtained at a specific concentration, and exceeding that concentration only serves to increase the freezing temperature. This minimum temperature is referred to as the eutectic temperature, and the corresponding solution concentration as the eutectic concentration. Changing the solution concentration to more or less than the eutectic concentration results in a higher freezing temperature and reduces the effectiveness of the brine as a deicing/anti-icing agent. When weather conditions permit, brine solution strengths should be reduced to achieve the lowest freezing point needed for the pavement and ambient temperatures. Reducing the solution concentration will also reduce salt use and therefore the cost of operations.

A study by Sutter (2008) indicated that, regardless of the distress mechanism, reducing the solution concentration reduces the magnitude of distress and the distress progression rate. Therefore, any effort to reduce deicer application

Table 4-1. Salt Brine Freezing Points

Salt	Concentration (wt%)	Freezing Point (F)
Sodium Chloride	23.30	-6.16°
Magnesium Chloride	21.00	-24.50°
Calcium Chloride	30.22	-57.60°
Calcium Magnesium Acetate	32.50	-17.50°

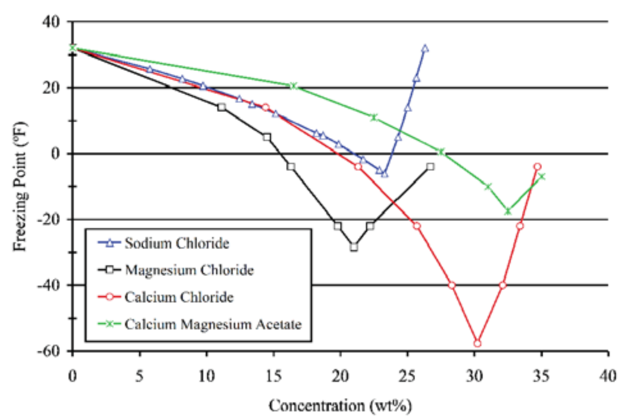


Figure 4-2. Partial phase diagrams for mixtures of water and common deicing/anti-icing salts. The freezing point of the water-salt solution (brine) is plotted as a function of the concentration of salt, in weight percent. Note that for all salts, a minimum freezing point is shown (Sutter 2008)

concentrations or rates will result in less damage to concrete structures. Note in Figure 4-2 that, as the brine solution concentration decreases to a relatively lower value (i.e., less than 5 percent by weight), the freezing points of the solutions become quite similar. This means that, as the brines become more diluted (which occurs as snow and ice melt), the relative difference in effectiveness (i.e., freezing point) diminishes. Therefore, using higher melting point solutions (i.e., brines of $MgCl_2$ and $CaCl_2$) at lower temperatures accomplishes little in terms of maintenance, yet it exposes the concrete to the chemicals most known to cause deterioration.

Obviously, there are significant trade-offs that must be evaluated when selecting a deicing/anti-icing chemical. It is important to keep in mind the negative effects of the various chemicals when making that selection, not just their relative performance as a deicer (see chapter 2). The study by Sutter (2008) indicated that NaCl brines have a minimal effect on hydrated cement paste, while showing a clear mechanism for chemical attack of hydrated cement paste in concrete for $MgCl_2$ and $CaCl_2$. It is recognized, however, that NaCl is corrosive to steel and therefore is not benign when used as a deicing chemical. Additionally, it is recognized that $MgCl_2$ and $CaCl_2$ are clearly more effective at lower temperatures and should be used only when such performance is required.

What about CMA?

Calcium magnesium acetate (CMA) solutions are not recommended for deicing purposes in Iowa.

Maturing period

It is recommended that newly placed concrete not be exposed to deicing chemicals for at least 30 days (Kosmatka and Wilson 2016). Even longer periods are required when the concrete contains SCMs, given the slower hydration rate associated with the use of SCMs.

This requirement is often overlooked, and concrete is exposed to deicing/anti-icing chemicals before it has adequately matured. The net result is usually scaling damage, but it is not out of the question that joint deterioration can also be affected.

Some jurisdictions stipulate a minimum curing period before applying any deicing chemicals to new concrete on residential streets.

CHAPTER 5. TREATMENT OF PAVEMENTS WITH JOINT DETERIORATION

This chapter briefly describes four potential treatments for pavements that are exhibiting joint deterioration: surface sealers, partial-depth repairs (PDRs), full-depth repairs (FDRs), and overlays. It also describes when it may be appropriate to recycle a pavement. For more detailed information on any of these treatments, consult the *Concrete Pavement Preservation Guide* (second edition) (Smith and Harrington (2014).

Selection and Timing of Repairs

When premature joint deterioration is observed in transverse and/or longitudinal joints, an evaluation of the causes and the rate of deterioration is critical for selecting the most cost-effective repairs and their optimum timing. The asset management philosophy of "selecting the right treatment, at the right time, at the right cost" definitely applies to joint spalling, because spalling can move from a relatively low-cost repair to a major cost repair in a relatively short time period.

Figure 5-1 (page 20) is a recommended decision matrix for selecting and timing preservation/rehabilitation techniques to help prevent and/or repair spalling in transverse and longitudinal joints and cracks. Use this matrix to indentify the joint condition you have and then take the appropriate action.

Address Underlying Causes

When implementing a repair technique for joint spalling, it is also important to mitigate the underlying cause(s) of the deterioration in order to avoid recurrence. As described in chapter 2, premature joint deterioration is likely to be the result of two conditions: (1) saturated joints with a moderate-to-poor air-void system and/or (2) the formation of oxychloride in the joint resulting from the reaction of cement with certain deicing chemicals. Mitigation strategies, therefore, could include the following:

- Reducing water access to the concrete
- Improving drainage for the concrete pavement system and for the joints in particular
- Selecting less reactive deicing/anti-icing salts for winter maintenance activities
- Deploying winter roadway maintenance systems that reduce the amount of applied salt to the lowest effective application rate.

Surface (Penetrating) Sealers for Existing Concrete Pavements

As noted previously, the use of concrete surface sealers has the potential to reduce joint damage (see chapter 3). Some jurisdictions are applying penetrating surface sealers to joints displaying shadowing or light spalling to mitigate or slow joint deterioration.

Partial-depth Repairs (PDR)

Partial-depth repair (Figure 5-2) is a time-tested method for the repair of spalling joints and cracks in concrete pavements. During the last 15 years, PDR methodology, materials, and equipment have substantially improved. When applied at appropriate locations, PDR has proven to be more cost effective than FDRs.

The primary limitation of using PDR is the depth of the repair. Partial-depth repairs should be in the top half of the pavement depth to achieve the intended service life of 10 to 15 years.

Typically, before spalling begins to appear, shadowing can be seen on each side of the joint, indicating that the joint is damaged. In cold-weather states, it is recommended that the joint be drained, any backer rod removed, and the saw cut filled.

When spalling starts to occur and reaches a width of 1 in. measured to the face of the joint or no more than 2 in. total width (refer to Figure 5-1), the spall should be filled with an asphalt rubber sealant and a PDR should be completed as soon as possible, preferably within a two-year window. (Even at low severity levels, cores should be taken to ensure that the deterioration is limited to the top one-third to one-half of the pavement thickness.)

When spalling reaches a width of 2 in. measured to the face of the joint or no more than 4 in. total width, the severity has reached a point requiring immediate PDR. (If spalling has extended below the limits of PDR, however, FDRs are required.)

Note: If measurable spalling is evident in pavements less than seven years old and the concrete mortar in the joints shows evidence of flaking, a petrographic analysis should be done to determine the air-void spacing and volume in the concrete at the joints. If the air-void system is poor and it is not possible to drain the saturated joints, then a PDR is not appropriate. Historically, pavements in this condition and age show continued deterioration adjacent to a PDR. Full-depth repairs are required under this condition, but they may have a limited service life.

1

Condition: Shadowing adjacent to joint

Action:
- Drain joint via subdrains
- Remove backer rod
- Minimize use of calcium chloride and magnesium chloride for deicing

2

Condition: Spalling up to 1 in. from face of joint or no more than 2 in. total width

Action:
- Same as no. 1
- Take cores to determine depth of deterioration
- Temporarily fill spalled area with sealant or asphalt patch
- Program partial-depth repair within two years

3

Condition: Spalling up to 2 in. from face of joint or no more than 4 in. total width

Action:
- Same as no. 1
- Complete partial-depth repair
- If cores show evidence of flaking, complete petrographic analysis to determine air voids and spacing; if poor air system, go to full-depth repair
- If pavement < 7 yr old, go to full-depth repair

4

Condition: Joint spalling > 4 in. from face of joint

Action:
- Take cores to determine depth of deterioration
- If deterioration is < T/2, complete partial-depth repair
- If deterioration is > T/2, complete full-depth repair
- If nearly every joint has severe spalling > 6 in. wide, see no. 5 and no. 6

5

Condition: Severe deterioration at every joint; no mid-panel deterioration; minimal vertical restrictions

Action:
- Mill each joint, remove loose material, and backfill with mortar
- Construct unbonded overlay

6

Condition: Severe deterioration at every joint, with vertical restrictions and/or where milling of the pavement would be problematic

Action: Recycle pavement and construct new pavement

Figure 5-1. Joint condition matrix

With a PDR, it is very important to re-establish the pavement joint with the same opening dimension as the in-place adjacent joint. This prevents point loading and compression failure of the repair. It is also important to ensure a good bond between the existing concrete and the repair material to prevent water infiltration. After the PDR is accomplished, diamond grinding (Figure 5-3) should be considered to produce a smooth riding surface and to restore the International Roughness Index.

Full-depth Repairs (FDR)

Full-depth repair is the preferred method for repairing higher severity joint and crack spalling. When spalling has reached a width greater than 4 in. measured to the face of the joint, it is much more likely that the depth of the deterioration exceeds half the pavement thickness. When this occurs, PDR is typically not a successful long-term repair and FDR should be considered. Again, the pavement needs to be cored to determine the depth of spalling and the existence/exposure of dowel bars.

Although FDRs can be designed and constructed to provide good long-term performance (see Figure 5-4), the performance of FDRs is very much dependent on their appropriate application and use of effective design and construction practices. Many FDR performance problems can be traced back to inadequate design (particularly poor load transfer design), construction quality, or the placement of FDRs on pavements that are too severely deteriorated.

Figure 5-2. Milled joint for removal of spalling (left photo) and finished PDR (right photo, courtesy of City of West Des Moines, Iowa)

Figure 5-3. Diamond-ground pavement surface (photo courtesy of The Transtec Group)

The cost efficiency of an FDR depends largely on the frequency of transverse and longitudinal joint spalling. For example, if every joint in a pavement section does not require FDR, then FDR is a strong candidate. If nearly every joint in a pavement section requires FDR, however, a cost analysis is recommended to determine if full-panel replacement or the construction of an unbonded concrete overlay would be a more appropriate rehabilitation method.

Unbonded Concrete Overlay of Concrete

An unbonded concrete overlay is an appropriate major rehabilitation technique for a concrete pavement with high-severity spalling. The deteriorated joints can be milled to remove loose material and filled with lower quality concrete or flowable mortar, eliminating expensive pre-overlay repairs. The joints do not have to be re-established because reflective cracking is not an issue with well-designed and constructed unbonded overlays.

Figure 5-5 shows a concrete pavement in Missouri with severe joint spalling. After careful examination of the pavement, it was decided to construct an unbonded concrete overlay. The

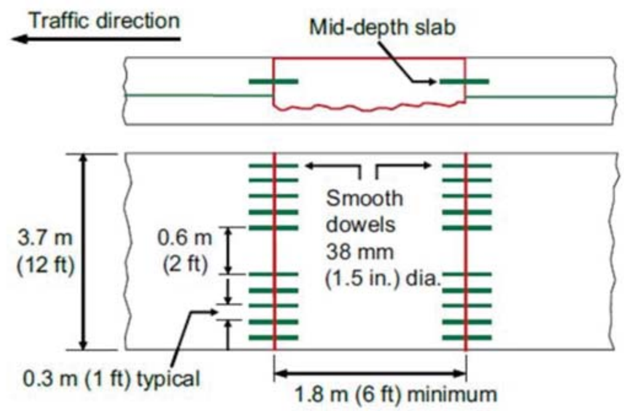


Figure 5-4. Example of FDR (plan view showing dowel bar layout)

Figure 5-5. Concrete pavement with severe joint spalling (photo courtesy of Todd LaTorella, MO/KS ACPA)

spalled joints were milled and patched with a flowable mortar to fill the void (Figure 5-6). Bonding agents were not used for the patch, and the joints were not re-established.

A separation layer consisting of either a nonwoven geotextile fabric or thin hot-mix asphalt interlayer is placed between the existing concrete pavement and the new unbonded concrete overlay (Figure 5-7). The separation layer prevents bonding and thus eliminates reflective cracking.

Recycling Concrete Pavements

In some instances in which pavement deterioration is beyond the state of feasible repair with PDR, FDR, or unbonded concrete overlay, appropriate option is reconstruction in place. Recycling concrete pavements into a base for the new pavement has proven to be a successful option. Recycled concrete aggregate (RCA), as a rule, is a high-quality material for unbound or stabilized applications such as bases compared to conventional new aggregate. An RCA has a rougher surface texture, higher shear strength, higher rutting resistance, and higher resilient modules (Van Dam et al. 2012) (see Figure 5-8).

Figure 5-6. Pre-overlay repair of spalled joint (photo courtesy of Todd LaTorella, MO/KS ACPA)

Pavements with a materials-related distress, such as alkali-silicate reaction (ASR), D-cracking, or freeze-thaw distress, have been effectively used as an unbound base material. It is recommended, however, that testing be performed to determine the severity of ASR in accordance with the FHWA's *Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silicate Reaction (ASR) in Transportation Structures* (Fournier et al. 2010).

Figure 5-7. Completed unbonded concrete overlay (photo courtesy of Todd LaTorella, MO/KS ACPA)

Figure 5-8. Concrete pavement for recycling (photo courtesy of Michigan State University)

CHAPTER 6. SPECIFICATION GUIDELINES

To provide for a successful restoration of deteriorated joints, it is necessary to define the proper materials and methods of repair or restoration. This chapter provides specification guidelines, including choosing the appropriate concrete mixture, and discusses the critical elements of constructing PDRs and FDRs. In this section, references are made to the Iowa DOT and Statewide Urban Design and Specifications (SUDAS) for both rural and urban applications.

Definitions

The following definitions are given for concrete mixtures and are further defined in Iowa DOT Materials IM 529:

C-mixes are the normal paving mix used in conventional paving. The numbering behind the C indicates a variation of fine and coarse aggregate. A C4 mix is composed of 50 percent fine and 50 percent coarse aggregates.

C2 is composed of 40 percent fine and 60 percent coarse
C3 is composed of 45 percent fine and 55 percent coarse
C4 is composed of 50 percent fine and 50 percent coarse
C5 is composed of 55 percent fine and 45 percent coarse
C6 is composed of 60 percent fine and 40 percent coarse

Class C-SUD and CV-SUD mixes are used on SUDAS projects in which higher durability is desired to reduce joint deterioration due to deicing chemicals. These mixes are designed with a lower w/c ratio to reduce permeability.

QM-C mixes are contractor- designed aggregate proportioning mixes for paving. Basic w/c ratio is 0.40. Maximum w/c ratio is 0.45. These mixes are used on Iowa DOT paving projects greater than 50,000 square (sq) yd.

Concrete Mixes

It is typical for different jurisdictions within the state of Iowa to specify different concrete mixes. For both urban and rural pavement applications, both standard and more durable mixes are used. On a typical paving project that includes a standard mix, Class C mixes are often specified. If a special durability mix is desired, a C-SUD/CV-SUD mix is specified. Tables 6-1 and 6-2 summarize the various concrete mix types utilized by Iowa DOT, municipalities, and counties based on standard mixes and higher durability mixes.

Iowa DOT Durability Mixes

The Iowa DOT has a QM-C (quality management concrete) mix that produces concrete with improved placement characteristics that is more readily slip-formed. Research has shown that mixes with improved workability can produce a more durable pavement. The QM-C mix is required on QM-C paving projects typically larger than 50,000 sq yd. The use of QM-C and an explanation of the materials are included in Chapter 9.6 of the *Iowa DOT Field Inspection Manual*. The mix is typically used on rural projects that require minimal handwork. The aggregate used is based on an optimized gradation of coarse, intermediate, and fine (sand) sizes. QM-C mixes are designed for use in slip-form paving operations only. The optimized gradation allows for an easier slip-form placement without edge slump, especially on thicker pavements. Most ready-mix plants are not equipped to batch three aggregates; therefore QM-C mixes are often batched from a central plant. Because there is additional effort for the mix design, increased amount of course aggregate, and field testing in QM-C mixes, Class C concrete is more economical on smaller projects, urban projects, and projects requiring extensive staging.

This mix is defined in Developmental Specification DS-15038 Quality Management Concrete (QMC). These mixes have aggregate proportioning designed by the contractor. The developmental specification (DS) includes important

Table 6-1. Mixes for Rural Application

Type	Iowa DOT	Cities	Counties
Standard Mixes	Class C Mixes	N/A	Class C Mixes
Durability Mixes	QM-C Mix	N/A	QM-C or C-SUD/CV-SUD

Table 6-2. Mixes for Urban Application

Type	Iowa DOT	Cities	Counties
Standard Mixes	Class C Mixes	Class C Mixes	N/A
Durability Mixes	QM-C[1] or C-SUD/CV-SUD	C-SUD/CV-SUD	N/A

1The Iowa DOT requires QM-C mixes on paving projects larger than 50,000 sq yd. In urban applications; QM-C mixes are considered for freeway projects but typically not specified where staging challenges exist.

guidance on mix design. A couple of points are notable in relation to durability and workability. First, the DS refers to IM 532 for various combinations of aggregate gradations. The DS provides further guidance to develop a mix based on gradations that fall within Zone II of the table for workability versus coarseness. Refer to Figure 6-1.

Second, the DS indicates a maximum w/c ratio of 0.42 and minimum 560 lb/yd^3 of cementitious content (Table 6-3).

The DS guides users to employ the workability/coarseness table as the primary method for selection of gradations. It points out that three different methods should be used to arrive at the optimum gradation. Along with workability/coarseness relationship, the remaining two methods are the power curve and the percent-retained chart. As pointed out in the DS, optimized gradations typically lead to a lower w/c ratio.

Municipality Durability Mixes

Municipalities in Iowa typically specify concrete mixes in the following way. If the city does not have joint deterioration concerns, then they often continue using their default Class C mix. SUDAS Section 7010 refers to Iowa DOT IM 529 and simply indicates that cities should use Iowa DOT Class C (or M) mix as described.

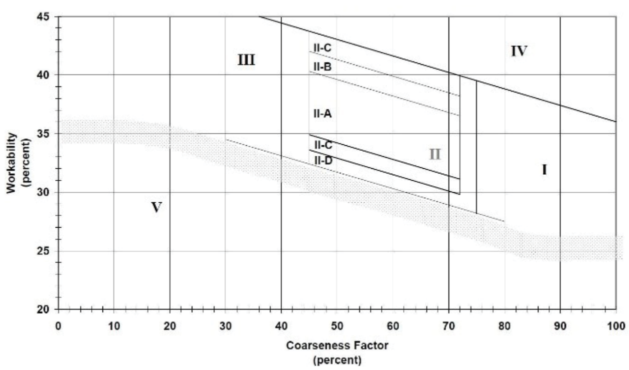


Figure 6-1. Workability factor vs. coarseness factor for combined aggregate

If the city has concerns with joint deterioration, then they should consider one of two high-durability mixes: C-SUD or CV-SUD. These mixes are identified in Iowa DOT IM 529. The C-SUD and CV-SUD mixes have the SUD designation for SUDAS. The "V" designation refers to the use of a Class V aggregate as specified in Iowa DOT Section 4117. While a conventional Class C concrete has a target w/c ratio of 0.43 and max of 0.488, the C-SUD mix has a target of 0.40 and max of 0.45. As a result, a C-SUD mix provides a more durable concrete pavement with lower permeability than conventional Class C mix due to its lower w/c ratio. With slip-form paving operations, however, the w/c ratio will typically be lower than the target w/c ratio during production with any mix design. The higher durability is desired to reduce joint deterioration due to the use of deicing chemicals. A joint saturated with de-icing salts limits the ability to allow the joint to dry out. To maintain the lower w/c ratio, a low- or mid-range water reducer may be necessary. For even greater durability, the use of ternary mixes should also be considered. Further references for mix proportioning in C-SUD and CV-SUD are included in Tables 6-4 and 6-5 (from Proportion Table 4 of Iowa DOT Materials IM 529). Note that these mixtures assume a basic w/c ratio of 0.40 and a maximum w/c ratio of 0.45.

Further references for mix proportioning in C-SUD and CV-SUD are included in Tables 6-4 and 6-5 (from Proportion Table 4 of Iowa DOT Materials IM 529). Note that these mixtures assume a basic w/c ratio of 0.40 and a maximum w/c ratio of 0.45.

The above mixture is based on Type I or Type II cements (Sp. G. = 3.14). Mixes using blended cements (Type IP or IS) must be adjusted for cement gravities listed in IM 401. Use proportions listed above if not utilizing three aggregates.

The above mixture is based on Type IP cements.

High Durability Mix

If a municipality desires a mix with an even higher durability, consideration should be given to supplementing higher amounts of Class C fly ash. This type of mix was developed for use on several street sections in West Des Moines. A report on the project (Wang et al. 2016) documents materials, test methods, and results of five different slip-form pavements using a highly durable mix to prevent premature joint deterioration. The mix does not meet IM 529 because it is over the

Table 6-3. Concrete Mixture Constraints

Nominal Maximum Coarse Aggregate Size	Greater than or equal to 1 inch
Gradation	Materials I.M. 532
Cementitious Content	Minimum, 560 pounds per cubic yard
Fly Ash Substitution Rate	See Article 2301.02, B, 6
Water/Cementitious Ratio	Maximum, 0.42
Air Content	6% ± 1%, Design Absolute Volume = 0.060
28 Day Flexural Strength, Third Point	Minimum, 640 pounds per square inch

Table 6-4. Mixes Using Iowa DOT Specifications Article 4110 and 4115 Aggregates

Mix Proportions (basic absolute volumes of materials per unit volume of concrete)					
Mix No.	Cement	Water	Air	Fine Aggregate	Coarse Aggregate
C-SUD	0.106	0.133	0.060	0.315	0.386

Table 6-5. Mixes Using Iowa DOT Specifications Article 4117 (Class V) Aggregates with Limestone

Mix Proportions (basic absolute volumes of materials per unit volume of concrete)					
Mix No.	Cement	Water	Air	Fine Aggregate	Coarse Aggregate
CV-SUD	0.114	0.135	0.060	0.379	0.311

20 percent fly ash limited assigned by SUDAS 7010 and Iowa DOT 2301. It was based on a similar mix used successfully by the Minnesota DOT. In short, it included a substitution rate of approximately 33 percent of Class C fly ash. In addition, because construction was scheduled in late fall, it required a minimum of 400 lb of portland cement per yd^3 to help counter cold-weather paving challenges such as delayed set times. Although the West Des Moines report refers to the mix as M-QMC, it is referred to in this document as C-SUD-C33 to avoid confusion with an Iowa DOT mix.

The intent for its use is for municipalities. This requirement was put in place to promote strength gain through heat generation during late-season paving. This mix is only recommended for circumstances where the highest level of durability is desired. The test data shows that the C-SUD-C33 mix is performing as intended, including improved resistance to deicing salts.

While conventional Class C mixes are successful in most paving applications, in order to provide for higher durability for the restoration of joints and cracks from early deterioration, it is recommended to use C-SUD (or CV-SUD) mixes. The current Iowa DOT and SUDAS specifications provide technical information on these mixes.

Partial-depth Repairs

In order to properly restore concrete joints that have early joint deterioration, reference should be made to Iowa DOT DS-15022 (Partial Depth PCC Finish Patches). It defines the materials and methods of construction for PDRs. The specification section defines three types of patches.

Types of patches

Finish patches

Square or rectangular in shape, less than 6 ft long when placed on joint or random crack.

Joint and crack repair patches

Square or rectangular in shape, placed on a longitudinal or transverse joint or random crack, and minimum of 6 ft long

Overdepth patches

Typically irregular in shape and placed to full depth of existing pavement in areas of unsound concrete. A bent tie bar is inserted into sound concrete to hold the patch at mid-depth.

The three types are illustrated in Figure 6-2 (on page 26).

Patch materials

Iowa DOT DS-15022 also defines three types of concrete patch materials—Class A, Class B, and Class C—which are defined in the following paragraphs.

Class A

Class A is a modified portland cement type to provide rapid-set and high early strength. Reference is given to Iowa DOT IM 491.20 for rapid-setting concrete patching materials. When a mortar is furnished, special consideration should be given to the manufacturer's recommendation for quality of concrete aggregate. This also includes use of pea gravel (minimum Class 2 durability) meeting the requirements of Iowa DOT Specification Section 4112.

Class B

Class B is defined as a high early strength mix (five-hour) meeting the requirements of IM 529 with the following requirements:

- Use Class M mix with calcium chloride (no fly ash)
- Place concrete within 30 minutes after introduction of calcium chloride
- For coarse aggregate, use crushed carbonate stone chips or pea gravel meeting the requirements of Iowa DOT Specification Section 4112.

Class C

Class C is a material with an early set that will allow opening to traffic in 24 to 36 hours. Coarse aggregates should meet requirements for Class B patching material. Class M concrete mix is used without calcium chloride.

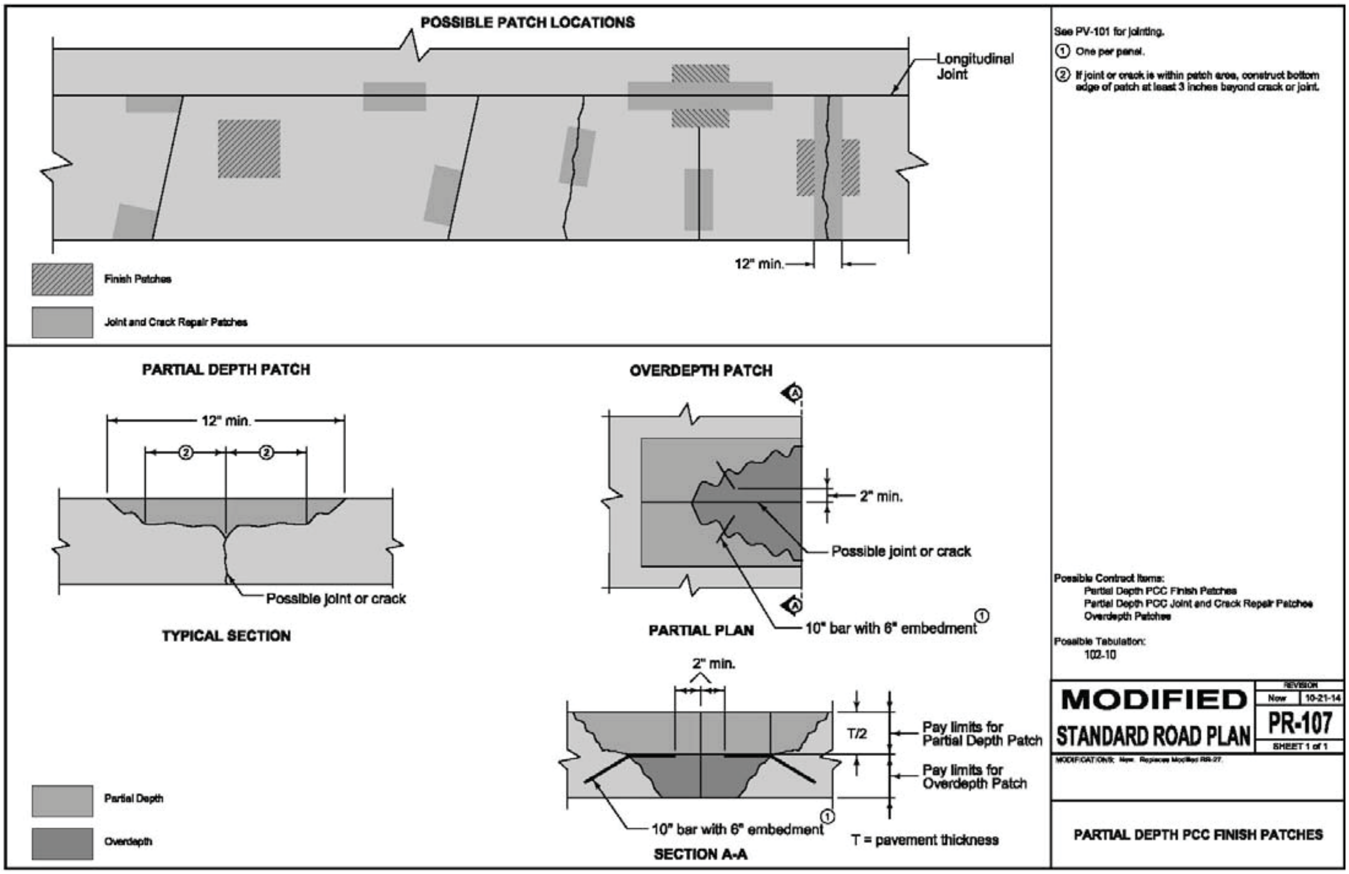


Figure 6-2. Modified Standard Road Plan PR-107

Proprietary mixes

Proprietary mixes are also available for use as a patch material. Various types are discussed in Chapter 5—Partial Depth Repairs of the *Concrete Pavement Preservation Guide* (second edition) (Smith and Harrington 2014). With any proprietary mix, batching and placement should follow the manufacturer's recommendations.

Joint boards/compressible inserts

Of critical importance is the re-establishment of the joint or crack by use of a joint board (Figure 6-3). The material is a resilient filler, cellulosic fiber, paraffin-coated cardboard, or other nonabsorbent, compressible material of proper shape. For longitudinal and transverse joints, one-piece boards will not be required in lengths exceeding 6 ft. For joints and open transverse cracks, board width shall be a nominal width of 0.25 in. It is important to extend the board into the pavement to the bottom of the patch. The joint board shall extend 3 in. beyond the patch if located within the slab.

The *Concrete Pavement Preservation Guide* (second edition) (Smith and Harrington 2014) provides a thorough discussion on PDRs in Chapter 5. It includes several items, including materials and construction considerations.

Figure 6-3. Joint board

Full-depth Repair

For joint deterioration that has advanced beyond a stage that is correctable by PDR, FDR should be provided. The Iowa DOT refers to this work as "Full Depth PCC Finish Patches."

Figure 6-4 is the Standard Road Plan PR-103 for full-depth finish patches with dowels. The Iowa DOT includes Standard Road Plans PR-101 through PR-105 for related technical details. Iowa DOT Specification Section 2529 covers Full Depth PCC Finish Patches for several applications including those listed here.

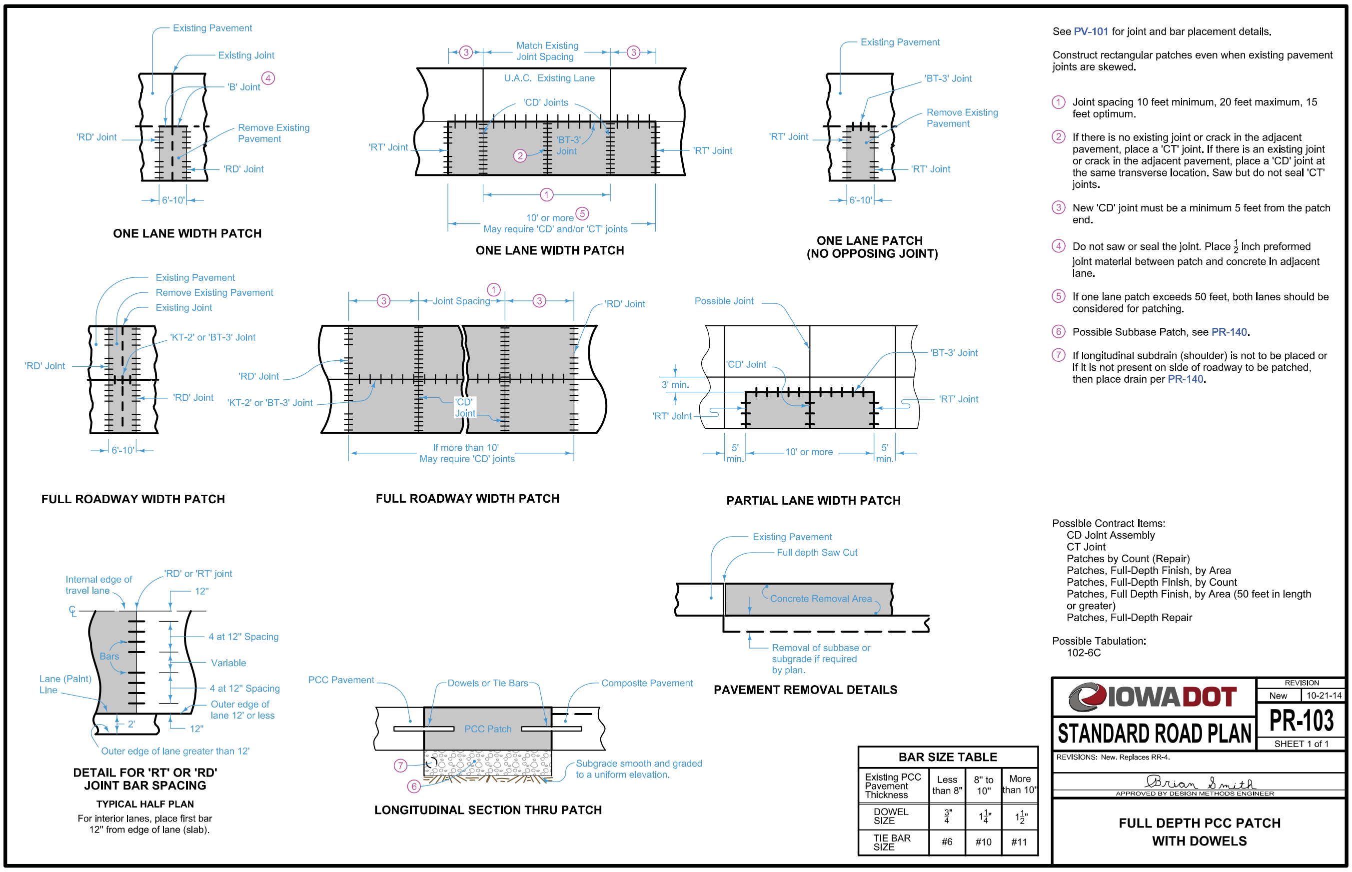


Figure 6-4. Standard Road Plan PR-103 for full-depth patches

Types of patches

- Full Depth PCC Finish Patches, without dowels
- Full Depth PCC Finish Patches, without dowels, composite section
- Full Depth PCC Finish Patches, with dowels
- Full Depth PCC Finish Patches, with dowels, composite section
- Full Depth PCC Finish Patches, continuously reinforced
- Full Depth PCC Finish Patches, continuously reinforced, composite section
- Full Depth PCC Finish Patches (50 ft or greater in length)

Patch materials

Refer to Section 2529 for placement, consolidation, finish, and curing of FDR. Special consideration should be given to smoothness (Section 2316) and joint sealing. Full-depth repair shall consist of a high early strength mix for early opening to traffic. Mix shall meet the requirements of Iowa DOT IM 529, with modifications as listed in 2529.02 of Section 2529.

Method of measurement and payment

The Iowa DOT's method of measurement and payment for FDR includes the combination of two bid items:

- By count (patches in each traffic lane are individually counted)
- By area (in sq yd)

"By count" includes sawing and installation of dowel bars at patch edges. "By area" includes the following:

- Removal of the old pavement
- Restoring the subgrade or subbase
- Furnishing and installing tie bars
- Restoring longitudinal reinforcement for continuously reinforced patches
- Furnishing and placing the patching material, including the asphalt binder, tack coat, curing, joint sealing, and placing backfill material in the disturbed area
- Profilograph testing and any required profile correction for patches 50 ft (15 meters) or greater in length

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