GUIDE FOR

Concrete Pavement Distress Assessments and Solutions

IDENTIFICATION, CAUSES, PREVENTION & REPAIR

IOWA STATE UNIVERSITY
Institute for Transportation

National Concrete Pavement Technology Center

OCTOBER 2018

About the National CP Tech Center

The mission of the National Concrete Pavement Technology (CP Tech) Center at Iowa State University is to unite key transportation stakeholders around the central goal of advancing concrete pavement technology through research, tech transfer, and technology implementation.

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16. Abstract

This guide was developed for transportation agency personnel and consulting engineers who are responsible for managing concrete pavement assets. Managing these assets includes monitoring pavement performance, developing project concepts, developing and administering pavement repair projects, and overseeing system maintenance.

Accurately identifying the distress, understanding the cause(s) of the distress and how to prevent it on future projects, and determining the proper repair procedures can be daunting and complicated. Selecting a pavement preservation strategy that does not address the root cause of a distress can result in wasted resources and additional inconvenience to the public.

The purpose of this guide is to clearly address these elements in a user-friendly, uncomplicated, and comprehensive manner. This guide incorporates proven and cost-effective solutions into a framework that assists in matching the appropriate solution(s) to a given distress. The chapters in this guide provide a detailed discussion of specific distresses that may need to be addressed.

This guide was developed as part of a Federal Highway Administration (FHWA) cooperative agreement to support more sustainable concrete pavement technical solutions.

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ABOUT THIS GUIDE

1. Overview

The *Guide for Concrete Pavement Distress Assessments and Solutions* is intended to assist with pavement preservation by helping to identify the causes and remedies for concrete pavement distress. By understanding the basic principles of concrete pavement preservation, engineers will be able manage their pavement networks to provide safe and dependable roadways while minimizing disruptions to the public for repair and maintenance activities.

Establishing a proactive approach to pavement condition monitoring and planned maintenance activities will reward owner agencies with not only long life from their concrete pavements but also reduced ownership costs and minimal disruption to the traveling public.

The number of failure mechanisms that may occur in concrete are fairly limited. However, most distress is a combination of more than one mechanism, exhibiting in an array of different forms. This manual goes into the details of the different types of distress observed in the field.

It is also important to remember that the presence of one or more distress type in a concrete pavement may not trigger corrective action by the engineer because such distresses may have limited impact on the pavement's overall functionality.

The authors and contributors to this manual have shared their wealth of knowledge and experience to help agencies achieve the goal of minimizing the cost of ownership of their concrete pavements.

2. Who this Guide is For

This guide was developed for transportation agency personnel and consulting engineers who are responsible for managing concrete pavement assets. Managing these assets includes monitoring pavement performance, developing project concepts, developing and administering pavement repair projects, and overseeing system maintenance. Users of this guide may include the following:

- Pavement inspectors and design engineers
- Field engineers responsible for developing project concepts
- Construction and maintenance staff responsible for administering contracts for repair of concrete pavement
- Asset management and pavement management engineers
- Consulting engineers

3. How this Guide was Developed

The National Concrete Pavement Technology (CP Tech) Center brought together leading national experts on concrete pavements in the engineering community to develop the technical content of this guide. In addition, a technical advisory committee with experience from Departments of Transportation (DOTs) and industry provided critical reviews and insights throughout development of this guide. The authors themselves have drawn upon their rich practical experience and lessons learned to make this a state-of-the-art resource for the management of concrete pavements.

4. Why this Guide was Developed

Pavements and their underlying support layers are a complex, interdependent system. The performance of this system is influenced significantly by traffic loading, climatic conditions, maintenance practices, the original design, and the construction of the pavement structure, foundation layers, and drainage system. Over the life of a concrete pavement, distresses can occur—and, in most cases, the distresses can be attributed to multiple causes.

Accurately identifying the distress, understanding the cause(s) of the distress and how to prevent it on future projects, and determining the proper repair procedures can be complicated. The purpose of this guide is to clearly address these elements in a user-friendly, uncomplicated, and comprehensive manner.

The need to cost-effectively manage pavement assets has become increasing more difficult and important. This is due to several factors. First, agency budgets are being stretched. Second, experienced staff is waning. Lastly, the public's demand to minimize disruptions while providing a safe riding surface is increasing.

Selecting a pavement preservation strategy that does not address the root cause of a distress can result in wasted resources and additional inconvenience to the public.

Historically, distresses in concrete pavements have been identified largely through visual surveys with limited investigation into the underlying cause(s) of the distress, and often with limited knowledge of how to cost-effectively maintain a concrete pavement in good condition. However, in the last few years, not only have significant technical advancements been made in distress assessment tools but more robust preservation treatment options have also been developed.

This guide incorporates proven and cost-effective solutions into a framework that assists you in matching the appropriate solution(s) to a given distress.

5. How to Use this Guide

This guide is intended to be used in combination with the Federal Highway Administration's (FHWA's) *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) in answering the following questions:

- Which distress is present?
- What caused it?
- How can it be prevented?
- Which repair options are available?

The chapters in this guide provide a detailed discussion of specific distresses that may need to be addressed. Each of the chapters is typically formatted as follows:

Chapter Number. Title of Distress

1. Description
    - Written summary of distresses
    - Images (with short description) of distresses

2. Severity
   - Opening dialogue
   - Table X. Summary of Severity of Distress
3. Testing
   - Field tests
   - Laboratory tests
4. Identification of Causes
   - Opening dialogue
   - Table of Physical and Material/Chemical Causes of Distress
5. Evaluation
   - Distress subject(s)
     - Cause
     - Prevention
   - Table X. Summary of Causes and Prevention of Subject Distress
6. Treatment and Repairs
   - Repair type(s) for specific repair and selection
   - Maintaining pavement subject
7. References

6. Organization and Scope of this Guide

Organization

This guide is organized into three divisions and addresses the following pavement types.

Division 1: Full-Depth Concrete Pavements

- Jointed Plain Concrete Pavement (JPCP)
- Jointed Reinforced Concrete Pavement (JRCP)
- Continuously Reinforced Concrete Pavement (CRCP)

Division 2: Concrete Overlays

- Bonded Concrete Overlay on Asphalt (BCOA)
- Bonded Concrete Overlay on Concrete (BCOC)
- Unbonded Concrete Overlay on Asphalt (UBCOA)
- Unbonded Concrete Overlay on Concrete (UBCOC)

Division 3: Laboratory and Field Testing

- All previously listed pavement types

Scope

Full-depth asphalt and composite pavements (hot-mix asphalt [HMA] over portland cement concrete [PCC]) are not addressed in this guide. However, there is a concept commonly called "buried treasure" where an existing asphalt resurfacing is removed from an underlying concrete pavement and is subsequently repaired and put back into service as a concrete pavement.

This guide does not address "buried treasure," but it may be a cost-effective solution for some composite pavements showing distress. Additional information can be obtained from the International Grooving and Grinding Association (http:/www.igga.net).

7. Where to Find Additional Information

Additional information on concrete pavement distresses can be found at the resources listed below.

- *Concrete Pavement Preservation Guide*
  http://www.cptechcenter.org/technical-library/documents/preservation_guide_2nd_ed_508_final.pdf
- *Distress Identification Manual for the Long-Term Pavement Performance Program*
  https://www.fhwa.dot.gov/publications/research/infrastructure/pavements/ltpp/13092/13092.pdf
- Guide for Partial-Depth Repair of Concrete Pavements
  http://www.cptechcenter.org/technical-library/documents/PDR_guide_Apr2012.pdf
- *Guide to the Prevention and Restoration of Early Joint Deterioration in Concrete Pavements*
  http://www.intrans.iastate.edu/research/documents/research-reports/2016_joint_deterioration_in_pvmts_guide.pdf
- *Integrated Materials and Construction Practices for Concrete Pavements: A State-of-the-Practice Manual*
  http://www.cptechcenter.org/technical-library/documents/imcp/imcp_manual_october2007.pdf

8. Reference Information for this Guide

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CHAPTER 1. INTRODUCTION TO DIVISION 1: FULL-DEPTH CONCRETE PAVEMENTS

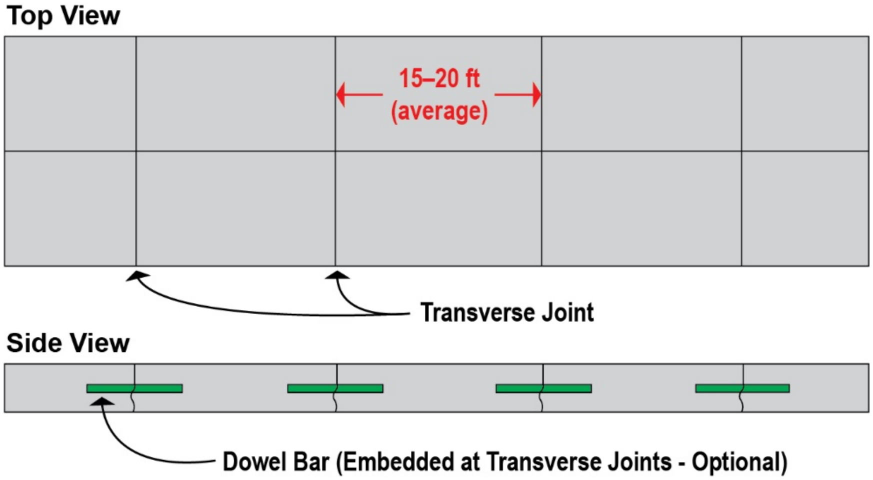
This chapter helps to identify where more detailed guidance on full-depth pavement distress can be found within Division 1 of this manual. A brief overview of each chapter is provided along with a photo of what this type of distress may look like.

When determining the best options for repairing a particular distress, it is important to have a general understanding of the characteristics of common full-depth concrete pavement types. Jointed plain concrete pavement (JPCP) and continuously reinforced concrete pavement (CRCP) each has its own design and performance characteristics. The following descriptions of these two pavement types are summarized below and were taken from the American Concrete Pavement Association (ACPA)'s Concrete Pavement Wikipave website.

Jointed Plain Concrete Pavement (JPCP)

JPCPs contain enough joints to control the location of all expected natural cracks. All necessary cracking occurs at joints and not elsewhere in the slabs. JPCP does not contain any steel reinforcement. However, there may be load transfer devices (e.g., dowel bars) at transverse joints and deformed steel bars (e.g., tie bars) at longitudinal joints. The spacing between transverse joints is typically between 15 and 20 feet for slabs 7–12 inches thick, as illustrated in Figure 1.1.

Figure 1.1 Jointed plain concrete pavement (JPCP)

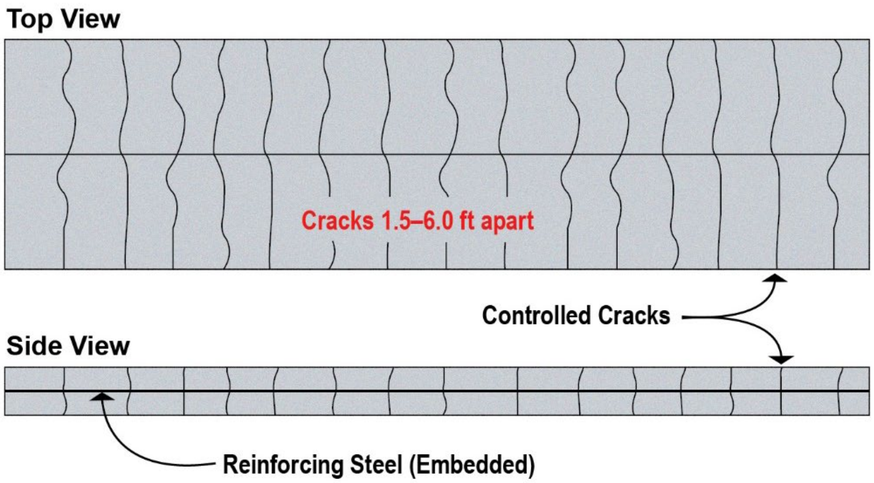


After ACPA

Continuously Reinforced Concrete Pavement (CRCP)

CRCP is a type of concrete pavement that does not require any transverse contraction joints. Transverse cracks are expected in the slab, usually at intervals of 1.5 to 6 feet, as illustrated in Figure 1.2.

Figure 1.2 Continuously reinforced concrete pavement (CRCP)



After ACPA

CRCP is designed with enough embedded reinforcing steel (approximately 0.6–0.7% by cross-sectional area) so that cracks are held together tightly. Determining an appropriate spacing between the cracks is part of the design process for this type of pavement.

Continuously Reinforced Concrete Pavement vs. Jointed Plain Concrete Pavement

CRCP designs generally cost more than JPCP designs initially due to increased quantities of steel. However, they can demonstrate superior long-term performance (with typical design service lives of 30 to 40 years) and cost-effectiveness. A number of state highway agencies choose to use CRCP designs in their heavy urban traffic corridors where traffic over the service life of the pavement can be on the order of tens of millions of equivalent load repetitions.

The full-depth pavement chapter topics and distresses are described below.

Chapter 2. Surface Defects

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Minor deformities or imperfections that are limited to the surface of a concrete pavement are often referred to as surface defects. These distresses typically do not significantly detract from the structural integrity of the pavement but can have an impact on its functional performance and its aesthetic appeal.

These surface defects include the following:

- Map cracking
- Plastic shrinkage
- Scaling
- Surface polishing/wear
- Popouts/mortar flaking

Chapter 3. Surface Delamination

Texas DOT

Surface delamination in concrete pavements is closely related in appearance to scaling and spalling. However, the mechanism of failure is different, as is discussed in detail in this chapter. Delamination may be viewed as the development of a horizontal crack within the slab that results in separation of the surface layer to a depth of 1/2 to 2 inches from the remaining concrete. Delamination may be limited to an extent, or it may be widespread depending on the basic cause of the separation. Although delamination is normally seen adjacent to pavement joints and may extend 3 feet or more into the slab, it can also occur anywhere in the slab.

Scaling, as discussed in Chapter 2 of this manual, has a similar appearance to delamination but does not extend beyond approximately 1/2 inch in depth. Compression spalling at joints or transverse cracks may also appear similar but is due to the intrusion of incompressible materials and is discussed in Chapter 8. Debonding of a bonded concrete on concrete overlay or separation of the lifts in two-lift paving is discussed in Chapters 15 and 16, and is not considered in this discussion of delamination.

Chapter 4. Material-Related Cracks

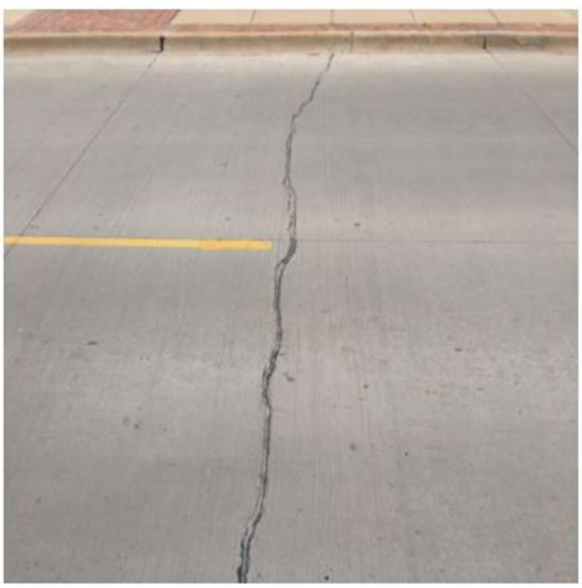
Randy Riley, Illinois Chapter, Inc. – American Concrete Paving Association

A material-related distress (MRD) is any failure that occurs in concrete pavements as the result of the properties of the materials in the pavement and their interaction with the environment. MRDs in concrete pavements are commonly typified by a network of multiple, closely spaced cracks, often accentuated with staining or deposits. However, visual inspection alone cannot confirm the presence or absence of material-related issues. Laboratory testing of pavement core samples is required to definitively confirm the mechanisms that may be contributing to the distress.

Material-related distresses include the following:

- D-cracking
- Alkali-silica reactivity (ASR)
- Alkali-carbonate reactivity (ACR)

Chapter 5. Transverse and Diagonal Cracking



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Transverse cracking, also called mid-panel or mid-slab cracking, is oriented laterally across the pavement and perpendicular to the pavement centerline. Diagonal cracking is oriented obliquely across a slab at roughly a 30- to 60-degree angle from the pavement centerline. Slab cracking may also develop longitudinally, in which the crack is oriented parallel to the pavement centerline. This chapter focuses on transverse and diagonal cracking, with Chapter 6 providing detailed information on longitudinal cracking.

Regardless of orientation, these types of cracks are differentiated from map cracking or other surface cracks (see Chapter 2) in that they are distinct cracks that typically extend through the entire thickness of the slab. Moreover, these cracks can also develop in conjunction with one another to produce what is often referred to as a shattered or broken slab (when the slab is divided into three or more pieces). Although cracking is perhaps the most common structural distress in concrete pavements, not all cracks are necessarily indicative of structural failures.

Chapter 6. Longitudinal Cracking

John Donahue, Missouri DOT

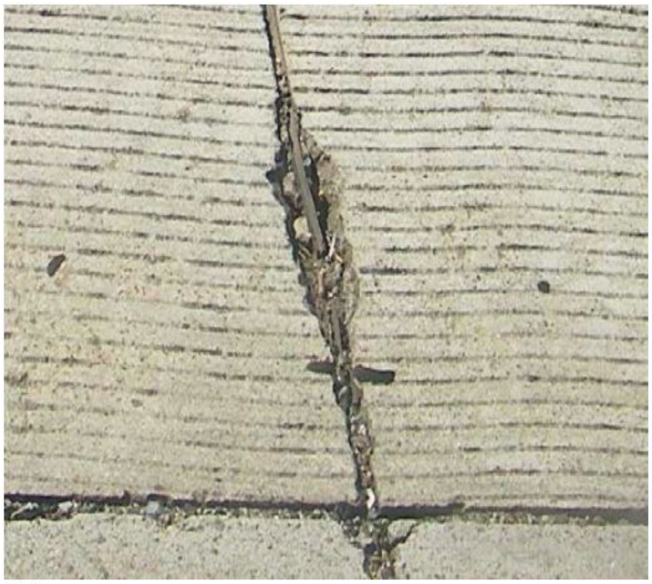
Longitudinal cracking is nearly parallel to the pavement centerline or lane-shoulder joint. Longitudinal cracks from surface cracks are covered in Chapter 2 and not in this longitudinal cracking chapter. This chapter covers full-depth cracks because they extend through the entire depth of the pavement. Other types of cracking include transverse, diagonal, and slab cracking—all of which are discussed in Chapter 5.

Chapter 7. Corner Cracking

Peter Taylor, National Concrete Pavement Technology Center

Corner cracking (also known as a “corner break”) is a distinct full-depth fracture in a jointed concrete pavement. Corner cracks intersect adjacent transverse and longitudinal joints at an angle of approximately 45 degrees with the direction of traffic. The lengths of the sides are rarely less than 1 foot, and are always less than half the width of the slab (by definition) on each side of the corner. Cracks with longer legs are considered diagonal cracks (see Chapter 5).

Chapter 8. Spalling—Transverse and Longitudinal Joints and Cracks



Caltrans

Transverse and longitudinal joint/crack spalling in concrete pavements is one of the most common, if not the most predominant, concrete pavement distress. Joint spalling is joint deterioration, which refers to cracking, chipping, or fraying of the concrete slab joint or crack edges of the transverse and longitudinal joints. Depending on the environmental conditions, spalling may develop predominantly in the top few inches of the slab or it may develop at a greater depth below the surface, eventually reaching full pavement depth.

Spalling problems include the following:

- Broken section of pavement adjacent to joints or cracks
- Shallow vertical drops
- Roughness

Chapter 9. Faulting

National Concrete Pavement Technology Center

Faulting is the difference in elevation across a joint or crack in a pavement due to loss of load transfer. It is a symptom of loss of uniform subgrade support. The loss of uniform support is due to pumping, which is the expulsion of soil and water due to traffic through a pavement joint, crack, or pavement/shoulder edge.

Chapter 10. Joint Curling and Warping

Daniel Frentress, Frentress Enterprises, LLC

Concrete slabs placed on grade undergo nonuniform volumetric changes due to temperature and moisture gradients. The temperature gradient changes throughout the day—the slab is normally colder on the top than the bottom from late at night through mid-morning, resulting in a negative temperature gradient. Under these conditions, the slab will have a tendency to undergo upward curling due to the lower surface temperature. As the top of the slab warms throughout the course of the day, its temperature becomes greater than that at the bottom, developing a positive temperature gradient. A downward curvature often develops as a result as the concrete at the surface expands.

Chapter 11. Blowups

Jeff Uhlmeyer, Washington State DOT

A blowup is a result of localized upward movement or shattering of a slab along a transverse joint or crack. Blowups often occur after heavy rainfall occurs followed by high temperatures, resulting in a high expansion buildup of pressure that can be dramatically released as the pavement thrusts upwards and/or shatters. Contributing factors are incompressible materials in the joint, a high coefficient of thermal expansion (CTE) of coarse aggregate as the concrete temperature increases, and long transverse joint spacing.

Chapter 12. Subgrades and Base Support Conditions—Categorized as Settlement and Heaves Distresses

Michael Ayers, Global Pavement Consultants, Inc.

Other than vehicle loading, the principal cause of distress in concrete pavements is volume change (and resulting movement) in either the concrete itself or in the underlying support system. The volume changes result in movement in the concrete that either exceeds the design parameters of the pavement when distresses occur, or that was not anticipated in the design. This chapter is devoted to the distresses in the concrete pavement due to volume changes in the subgrades and bases.

The distresses from volume change in concrete pavement from the subgrade and base typically show up as cracking as the result of settlements or heaves of the subgrade/base. Therefore, this chapter has been developed and formulated around settlement and heave categories since they do represent the majority of these types of distresses.

Chapter 13. Continuously Reinforced Concrete Pavement (CRCP)

Michael Plei

CRCPs are generally used for heavily trafficked roadway applications. These pavements differ from the more widely used JPCPs due to the presence of continuous longitudinal reinforcement ranging from approximately 0.70 to 0.80% of the cross-sectional area of the pavement slab.

CRCP distresses include the following:

- Punchouts
- Deep spalling
- Wide transverse cracks
- Longitudinal cracks

CHAPTER 2. SURFACE DEFECTS

1. Description

Minor deformities or imperfections that are limited to the surface of a concrete pavement are often referred to as surface defects. As described later in Table 2.1 (and illustrated in Figure 2.1), these defects can include items such as map cracking (also called crazing), plastic shrinkage cracking, scaling, surface polishing, surface wear, and popouts or mortar flaking. These distresses typically do not significantly detract from the structural integrity of the pavement but can have an impact on its functional performance and aesthetic appeal.

Figure 2.1 Common surface defects

a. Map cracking

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b. Plastic shrinkage cracking
National Concrete Pavement Technology Center

c. Scaling
National Concrete Pavement Technology Center

d. Surface polishing
Peter Taylor, National Concrete Pavement Technology Center

e. Surface wear in the wheel paths
Jeff Uhlmeyer, Washington State DOT

f. Popouts

2. Severity

Severity levels are typically not associated with surface defects. The measurement methods used for surface defects are summarized in Table 2.1 and modified from the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014, Taylor et al. 2007, and PCA 2001).

Table 2.1 Severity levels and measurements

Distress	Description	Measurements
Map Cracking (Crazing)	A network of cracks and fine fissures on the concrete surface that enclose small and irregularly shaped areas (side dimensions less than 2 in. [51 mm]). Larger cracks, if present, are frequently oriented in the longitudinal direction of the pavement and are interconnected by finer transverse or random cracks. These cracks are shallow, often only 0.125 in. (3 mm) deep. Map cracking may be localized or may occur over the entire surface of the concrete slab.	The number of occurrences and affected area (in ft^2 or m^2) is recorded. When an entire section is affected, it is considered a single occurrence.
Plastic Shrinkage Cracking	Shallow, closely spaced parallel cracks appearing at the surface, typically no more than about 1 to 2 in. (25 to 51 mm) deep that generally occur perpendicular to the wind direction.	The number of occurrences and affected area (in ft^2 or m^2) is recorded. When an entire section is affected, it is considered a single occurrence.
Scaling	Physical deterioration of the upper concrete slab surface, normally 0.1 to 0.5 in. (3 to 13 mm) may occur anywhere on the pavement.	The affected area (in ft^2 or m^2) is recorded.
Surface Polishing	Abrasive wear under traffic of the pavement surface texture and the aggregate, creating a smooth and polished surface.	The representative popout density or mortar flakes (in number per ft^2 or m^2) is recorded.
Surface Wear	Wearing away of the concrete and creation of “ruts” in the wheel path caused by studded tires or chains.	The depth of surface wear at specified intervals is recorded along the project.

Distress	Description	Measurements
Popouts or Mortar Flaking	Popouts are small fragments of pavement broken loose from the surface, normally ranging from about 0.5 to 2 in. (13 to 51 mm) in diameter and depth. A fractured aggregate particle or particle socket is typically found at the bottom of the depression where the popout has occurred. Surface voids in a concrete pavement resembling popouts may be the result of the presence and erosion of clay, mud, or other soft materials embedded in the concrete. Mortar flaking is the dislodging of small segments of mortar directly above coarse aggregate particles.	The representative popout density or mortar flakes (in number per ft^2 or m^2) is recorded.

Sources: Modified from Miller and Bellinger 2014, Taylor et al. 2007, PCA 2001

3. Testing

Depending upon the type of surface defect, a field or laboratory test may need to be completed. More detailed information regarding testing can be found in Chapter 19.

Field Tests

The field test used for crack surface defects is coring, which allows the user to have a visual representation of the pavement below the surface. The field test for friction characteristics of the surface is simply called friction testing.

Coring

Concrete core samples can be retrieved in the areas of the surface defects to determine the depth of the cracks and to look for the presence of any staining or exudate that may be suggestive of a material-related distress (MRD); see Chapter 4.

Friction Testing

Surface friction testing may be useful if significant polishing of the surface has occurred that may affect the frictional characteristics of the pavement. Surface friction testing is commonly performed using a locked-wheel trailer in accordance with ASTM E274, Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire.

Laboratory Tests

If MRD is suspected as the cause of the surface defect, laboratory testing will be required to confirm the mechanism. This testing can be performed on samples obtained from field coring, and should consist of petrographic examination conducted in accordance with ASTM C856.

4. Identification of Causes

Surface defects develop as a result of a variety of factors, most of which are related either to concrete materials, mixture aspects, or construction practices. Traffic impacts and maintenance practices also play a role in a few of the distresses (see Table 2.2).

Table 2.2 Physical and material or chemical causes of distress

Distress	Category	Description
Surface Defect (Material or Chemical)	Water-to-Cementitious Material (w/cm) Ratio	High w/cm ratio mixtures result in more bleed water and reduced surface strength due to an increase in capillary porosity.
Surface Defect (Material or Chemical)	Concrete Mix Design	Concrete mixtures containing high amounts of cement may be more susceptible to the development of surface defects, largely due to increased shrinkage.
Surface Defect (Material or Chemical)	Aggregate Characteristics	Concrete with a high volume of medium-sized aggregate pores (0.1 to 5 µm) that are easily saturated can absorb moisture and, upon freezing, can rupture the aggregate to create popouts (PCA 2001). Additionally, aggregates that contain shale, soft and porous materials (such as clay lumps or other deleterious materials), and certain types of chert may be prone to popouts.
Surface Defect (Material or Chemical)	Abrasion Resistance	The use of low abrasion-resistant aggregates can cause surface marring and wearing away of the concrete surface under traffic loading.
Surface Defect (Material or Chemical)	Alkali-Aggregate Reaction (AAR)	AAR refers to two different types of deleterious reactions that can occur between cement and aggregates, which can lead to map cracking and scaling of the concrete surface. Ultimately, this leads to spalling, blowups, and other pressure-related damage (see Chapter 4). AAR is often a slow process and the deterioration typically takes several years to progress.
Surface Defect (Material or Chemical)	Deicing Salts	The use of chemical deicers can accelerate the physical mechanisms responsible for freeze-thaw deterioration of concrete (ACI 2008).
Surface Defect (Physical)	Surface Finishing	Overworking of the concrete surface or finishing while bleed water is present on the surface can cause significant weakening at the surface and contribute to the formation of map cracking and scaling.

Distress	Category	Description
Surface Defect (Physical)	Absorption of Mixing Water Into the Aggregate	If adequate stockpile moisture control is not maintained, significant slump loss can occur as the mixing water gets absorbed into the aggregate, which can lead to finishing issues.
Surface Defect (Physical)	Curing	Late or inadequate curing impacts surface strength, abrasion resistance, and surface durability.
Surface Defect (Physical)	Rain	If the surface and edges of a newly placed concrete are not protected from rain, the surface can erode and wash away the paste; moreover, the additional water can elevate the surface w/cm ratio, which can lead to map cracking and scaling.
Surface Defect (Physical)	Weather	Rapidly changing weather conditions can affect the quality of the curing. When that happens, the durability and strength of the concrete surface is also affected.
Surface Defect (Physical)	Clay Balls	Clay balls generically refer to voids created in a concrete mixture as the result of clay, dirt, or other friable materials mixed into the coarse and fine aggregates at the batch plant. These voids may appear at the surface after traffic and environmental loadings.
Surface Defect (Physical)	Traffic	Traffic plays a role in the development of surface polishing and surface wear. Surface polishing is the result of traffic repetitions wearing away the surface texture and polishing the susceptible aggregate materials, creating a surface with reduced surface friction. Surface wear is caused by vehicles equipped with either studded tires or tire chains that mechanically abrade and erode the concrete, creating ruts in the wheel paths. Concrete constructed with softer, less wear-resistant aggregate is more susceptible to these forms of deterioration.

Source: Taylor et al. 2007, National Concrete Pavement Technology Center

5. Evaluation

The following summarizes the common causes and some of the potential prevention/mitigation strategies for the common surface defects identified in this chapter. Note that all of these surface defects can be affected by changing weather conditions, unique mix design characteristics, and deicing/anti-icing practices.

Map Cracking (Crazing)

Cause

Overworking/Overfinishing of Concrete Surface: While the finishing of most concrete pavements takes place prior to the appearance of bleed water, any overworking of the concrete surface while bleed water is present can cause significant weakening of the material at the surface of concrete and contributes to map cracking. Overworking the surface may also compromise the air void system and reduce the ability of the mixture to resist freezing and thawing. Cement that is sprinkled on the surface in an attempt to absorb bleed water concentrates fines at the surface; these fines will dry rapidly and shrink, leading to additional map cracking.

Late or Inadequate Curing: Curing can have a significant effect on the quality of the concrete surface. An approved curing compound should be applied as soon as possible to the surface and at the specified application rate to retain mix water to support hydration. Late or inadequate curing impacts surface strength, abrasion resistance, and surface durability.

Batching Absorptive Aggregates: When the concrete contains aggregate particles with a high volume of medium-sized pores (0.1 to 5 µm) that are easily saturated, they can absorb moisture and, when freezing occurs, the associated volumetric expansion can create enough pressure to rupture the aggregate and concrete surface and create popouts (PCA 2001). Additionally, aggregates that contain appreciable amounts of shale, soft and porous materials (such as clay lumps or other deleterious materials), and certain types of chert may be prone to popouts. Highly absorptive coarse aggregate (absorption of 3 percent or greater) must be batched at saturated surface dry (SSD) or wetter, or they will otherwise continue to absorb mix water, resulting in workability issues and potential map cracking.

Alkali-Aggregate Reaction (AAR): As further described in Chapter 4, there are two forms of AAR:

- Alkali-silica reactivity (ASR) is the most common form of AAR and is a chemical reaction between reactive silica in the aggregate and the alkalis present in the pore solution of the hydrated cementitious paste. The result of this reaction is the formation of a gel that expands in the presence of moisture and cracks the concrete matrix. ASR can result in map cracking of the concrete or popouts on the surface of the concrete pavement and can also contribute to cracking and spalling (see Chapter 4). If ASR is suspected on an existing concrete pavement, a petrographic evaluation in accordance with ASTM C856 should be conducted to confirm the distress mechanism.
- Alkali-carbonate reactivity (ACR) is the reaction between the alkalis in the pore solution of the hydrated cementitious paste and certain carbonate rocks, particularly dolomitic limestones that have a unique texture and the presence of clay. The reaction is highly expansive and can lead to deterioration of the concrete. Aggregates susceptible to ACR are less common and otherwise unsuitable for use in concrete (Taylor et al. 2007). If ACR is suspected, a petrographic evaluation in accordance with ASTM C856 should be conducted.

Prevention

Use Moderate Slump Mixtures with Low w/cm Ratios: Although typically not a problem for most concrete paving, high w/cm ratio mixtures result in more bleed water and reduced surface strength due to an increase in capillary porosity, which can contribute to the formation and propagation of surface defects. This may be more of a problem on small hand-pour placements that use high-slump mixes.

Use Durable, Nonreactive Aggregates: The use of durable aggregates is critical to the development of long-lasting concrete mixtures. Abrasion resistance is an important property related to the type and quality of the aggregates used, and the compressive strength of concrete. Generally, harder aggregates (such as granite or traprock) resist wear and abrasion better than softer aggregates (such as certain limestones), and proper mix design and curing will help to achieve adequate strength that will resist surface degradation. Abrasion resistance of aggregates is commonly measured using ASTM C131 or ASTM C535, while the wear resistance of concrete can be measured using several different methods (ASTM C418, ASTM C779, ASTM C944, and ASTM C1138). The use of ASR-susceptible aggregates should be avoided or mitigated using an appropriate supplementary cementitious materials (SCM).

Use SCMs or Blended Cements to Control ASR: Concrete mixtures incorporating suitable SCMs added to the mixture or that employ blended cements (mixtures of portland cement, fly ash, and slag cement) are effective means of controlling the deleterious alkali-silica reaction and minimizing the associated distress manifestations.

Design Concrete with Low Permeability: Dense concrete mixtures reduce the migration of water and other solutions (e.g., deicing chemicals) through the mixture, thereby minimizing the development of distresses related to those fluids. The introduction of fly ash into concrete mixtures is one way of reducing the permeability of concrete mixtures, along with the use of a reduced w/cm ratio and proper curing to help promote longer term hydration.

Keep Stockpiles Wet: If adequate stockpile moisture control is not maintained, significant slump loss can occur as the mixing water gets absorbed into the aggregate, which can lead to placement problems and finishing issues. Thus, it is important to keep stockpiles wet when absorptive aggregates are used.

Use Proper Finishing Practices: This consists of a number of key considerations, including the following:

- Avoid overworking the surface. Continuously working the surface may compromise the air void system and reduce the ability of the mixture to resist freezing and thawing.
- Refrain from spraying water on surface during finishing. Spraying water on the surface during finishing raises the w/cm ratio at the surface and creates a weakened layer of mortar that shrinks and leads to the formation of map cracking, scaling, and other surface defects.
- Avoid sprinkling cement on the surface to dry bleed water. Cement that is sprinkled on the surface in an attempt to absorb bleed water will dry rapidly and shrink, leading to additional map cracking.
- Avoid finishing while bleed water is present. Finishing while bleed water is present on the surface can cause significant weakening of the material at the surface of concrete and contribute to the formation of map cracking and other surface defects.

Use Effective Curing Practices: Effective curing techniques should be used as soon as possible after concrete placement. This can be accomplished through the use of an approved curing compound at the specified application rate or, when practical, through wet curing methods (fog spraying, wet burlap). Evaporation retarders, which are sprayable solutions that are applied to fresh concrete to reduce the rate of surface moisture evaporation prior to finishing, may also be used.

Plastic Shrinkage Cracking

Cause

Rapid Evaporation: Highly evaporative conditions (e.g., low humidity, windy conditions, and exposure to direct sunlight) contribute to rapid moisture loss from the surface, which can lead to the development of plastic shrinkage cracking in the concrete.

Prevention

Use Durable Mixes with a Low w/cm Ratio: Durable mixes with a low w/cm ratio will be less susceptible to shrinkage forces that could lead to the formation of plastic shrinkage cracking.

Minimize Cement Contents: Concrete mixtures containing high amounts of cement may be more susceptible to the development of surface defects, largely due to increased shrinkage.

Reduce Aggregate Absorption: Highly absorptive aggregates can remove water from the mixture that can lead to increased shrinkage.

Use Effective Curing Practices: As described previously, effective curing techniques should be used as soon as possible after concrete placement. This can be accomplished through the use of an approved curing compound at the specified application rate or, when practical, through wet curing methods (fog spraying, wet burlap). Evaporation retarders, which are sprayable solutions that are applied to fresh concrete to reduce the rate of surface moisture evaporation prior to finishing, may also be used.

Employ Proper Hot and Cold Weather Paving Practices (as appropriate). Paving under extreme weather events can lead to conditions that can contribute to rapid moisture loss and the development of plastic shrinkage cracking. The industry standards for paving under these conditions (ACI 2010, ACI 2016) should be followed, or, if possible, the concrete placement should be delayed until the extreme conditions subside.

Scaling

Cause

Freeze-Thaw and Deicing Chemicals: Scaling is the loss of surface mortar and is primarily related to the expansion of water in the concrete as it freezes. However, the use of chemical deicers can accelerate the physical mechanisms responsible for freeze-thaw deterioration of concrete (ACI 2008), and this has become more acute as many agencies have adopted aggressive snow and ice policies featuring the more frequent use of deicing brines placed at high concentrations just prior to snow events. A number of factors are at work, including the increased level of saturation that occurs when salt solutions are present, thermal shock that occurs due to the phase change of ice to water, an increase in osmotic pressures due to changes in pore solution chemistry, and the potential for salt crystallization within confined pore spaces (Mindess et al. 2003, ACI 2008, Kosmatka and Wilson 2016).

In the last decade, concerns have been raised regarding the potential for deleterious effects of certain chemical deicers on concrete, most notably magnesium chloride ($MgCl_2$) and calcium chloride ($CaCl_2$) (Sutter et al. 2008). Chemical deicers based on $MgCl_2$ and $CaCl_2$ are becoming increasingly popular due to their effectiveness over a broad range of temperatures (Sutter et al. 2008, Taylor et al. 2012, Weiss and Farnam 2015). They are commonly applied at relatively high concentrations as anti-icing agents prior to a winter precipitation event, a practice that can result in direct absorption of the solutions at full concentration into the pavement surface and joints. Such deicing chemicals may lead to the formation of expansive calcium oxychloride compounds, typically when temperatures are just above freezing. Distress is most commonly observed in the joint saw cuts where the solution gets trapped.

Improper Surface Finishing and Curing: While finishing of most pavements takes place prior to the appearance of bleed water, any overworking of the concrete surface, or finishing while bleed water is present on the surface, can cause significant weakening of the material at the surface of concrete and contribute to the formation of scaling. Furthermore, late or inadequate curing can affect surface strength and durability, which could lead to scaling.

Damage from Rain: When the surface and edges of newly placed concrete pavement are not protected from rain, the surface can erode, washing away the paste; moreover, the additional water at the concrete pavement surface can elevate the surface w/cm ratio (particularly if worked into the surface), which can lead to map cracking and scaling.

Prevention

Minimize the Use of Deicing Chemicals: Although not always possible, more limited use of deicing chemicals may reduce the potential for scaling, particularly on mixes with marginal durability characteristics. Furthermore, it is generally recommended that deicing chemicals should not be used on new concrete for at least 30 days after placement (ACI 2008).

Protect Slab from Rain: To address the potential for rain damage, a contractor should have plastic sheeting and steel side forms or wooden boards available at all times to protect the surface and edges of newly placed concrete (ACPA 2003).

Ensure Proper Air Void System: A proper air void system, featuring a uniform distribution of small air voids throughout the paste, will greatly improve the concrete's resistance to surface scaling.

Use Proper Finishing Practices: This consists of a number of key considerations, including the following:

- Avoid overworking the surface: Continuously working the surface may compromise the air void system and reduce the ability of the mixture to resist freezing and thawing.
- Refrain from spraying water on surface during finishing: Spraying water on the surface during finishing raises the w/cm ratio at the surface and creates a weakened layer of mortar that shrinks and leads to the formation of map cracking, scaling, and other surface defects.
- Avoid sprinkling cement on the surface to dry bleed water: Cement that is sprinkled on the surface in an attempt to absorb bleed water will dry rapidly and shrink, leading to additional map cracking.
- Avoid finishing while bleed water is present: Finishing while bleed water is present on the surface can cause significant weakening of the material at the surface of concrete and contribute to the formation of map cracking and other surface defects.

Use Effective Curing Practices: As described previously, effective curing techniques should be used as soon as possible after concrete placement. This can be accomplished through the use of an approved curing compound at the specified application rate or, when practical, through wet curing methods (e.g., fog spraying, wet burlap). Evaporation retarders, which are sprayable solutions that are applied to fresh concrete to reduce the rate of surface moisture evaporation prior to finishing, may also be used.

Surface Polishing or Surface Wear

Cause

Use of Aggregates with Poor Abrasion Resistance: The type and quality of aggregates is the primary factor that can lead to surface polishing in concrete pavements. Soft aggregates can abrade more quickly than others, leading to a polished surface that can lead to potential surface friction issues.

Exposure to Traffic: Traffic plays a role in the development of surface polishing and surface wear. Surface polishing is the result of traffic repetitions wearing away the surface texture and polishing the susceptible aggregate materials, creating a surface with reduced surface friction. Surface wear is caused by vehicles equipped with either studded tires or tire chains that mechanically abrade and erode the concrete, creating ruts in the wheel paths. Concrete constructed with softer, less wear-resistant aggregate is more susceptible to these forms of deterioration than concrete made with harder aggregates.

Improper Surface Finishing and Curing: Overworking or over-troweling of the pavement surface can produce an overly polished surface that could contribute to surface friction issues.

Prevention

Use Wear-Resistant Aggregates: As previously described, abrasion resistance is an important property related to the type and quality of the aggregates used and the compressive strength of concrete. Generally, harder aggregates (such as granite or traprock) resist wear and abrasion better than softer aggregates (such as certain limestones), and proper mix design and curing will help to achieve adequate strength that will resist surface degradation. Abrasion resistance of aggregates is commonly measured using ASTM C131 or ASTM C535, while the wear resistance of concrete can be measured using several different methods (ASTM C418, ASTM C779, ASTM C944, and ASTM C1138).

Minimize Moisture Loss: As described previously, it is important to employ effective curing techniques as soon as possible, through the use of an approved curing compound at the specified application rate or, when practical, through wet curing methods (fog spraying, wet burlap). Evaporation retarders may also be used as a way of maintaining surface moisture.

Use Concrete Mixtures with Adequate Strength: Overall concrete strength is an indicator of the abrasion resistance of the surface, so it is imperative that an appropriate mix design be developed and that proper curing procedures (see above item) be employed.

Use Proper Finishing Practices: This consists of a number of key considerations, including:

- Avoid overworking the surface. Continuously working the surface may compromise the air void system and reduce the ability of the mixture to resist freezing and thawing.
- Refrain from spraying water on surface during finishing. Spraying water on the surface during finishing raises the w/cm ratio at the surface and creates a weakened layer of mortar that shrinks and leads to the formation of map cracking, scaling, and other surface defects.
- Avoid sprinkling cement on the surface to dry bleed water. Cement that is sprinkled on the surface in an attempt to absorb bleed water will dry rapidly and shrink, leading to additional map cracking.
- Avoid finishing while bleed water is present. Finishing while bleed water is present on the surface can cause significant weakening of the material at the surface of concrete and contribute to the formation of map cracking and other surface defects.

Use Effective Curing Practices: As described previously, effective curing techniques should be used as soon as possible after concrete placement. This can be accomplished through the use of an approved curing compound at the specified application rate or, when practical, through wet curing methods (fog spraying, wet burlap). Evaporation retarders, which are sprayable solutions that are applied to fresh concrete to reduce the rate of surface moisture evaporation prior to finishing, may also be used.

Popouts or Mortar Flaking

Cause

Use of Unsound or Reactive Aggregates (Popouts): Near-surface aggregate particles with a high absorption and a relatively low specific gravity can absorb moisture, which can expand in freezing conditions and create pressures that cause the particle to fragment and dislodge at the surface (PCA 2001). And, as described previously, ASR-susceptible aggregates can react with alkalis in the cement to produce an expansive gel that could lead to popouts.

Dislodging of Mortar above Aggregate (Mortar Flaking): Excessive moisture loss and early drying of the surface mortar can be accentuated over aggregate particles located near the surface, creating a weak mortar layer and poor aggregate bond (PCA 2001). Upon freezing in a saturated condition, the thin, weakened mortar layer breaks away from the aggregate.

Erosion of Soft, Embedded Materials (i.e., Clay Balls): As indicated previously, clay balls is a generic term that describes voids created in a concrete mixture as a result of lumps of conglomerate materials consisting of clay, dirt, or other friable material being mixed into the concrete with the coarse or fine aggregates at the batch plant (ACPA 2004). The effects of clay balls may not be apparent at first but over time their presence may lead to the development of voids at the surface (see Figure 2.2) as a result of the clay particles absorbing water and expanding when frozen, or if traffic loadings break or crack the thin mortar-skin above the clay ball (ACPA 2004). The defect typically appears a few weeks or months after paving but in some cases may not become evident until after a winter season of freeze-thaw cycling.

Figure 2.2 Void created by clay ball at the concrete surface

Prevention

Use Durable Mixes with a Low w/cm Ratio: Mixtures with a low w/cm ratio will help reduce saturation potential and increase the resistance to expansive pressures. The mix should also contain a uniformly distributed air void structure.

Avoid the Use of Problem Aggregates: Aggregates with a history of popouts or ASR-susceptibility should be avoided to help minimize popouts. Beneficiation, or the removal of unwanted or deleterious materials from the aggregate, can also be considered for some aggregate sources but this may be cost-prohibitive.

Use SCMs or Blended Cements: This will help to minimize popouts that are the result of ASR-susceptible aggregates.

Use Proper Finishing Practices: This consists of a number of key considerations, including:

- Avoid overworking the surface. Continuously working the surface may compromise the air void system and reduce the ability of the mixture to resist freezing and thawing.
- Refrain from spraying water on surface during finishing: Spraying water on the surface during finishing raises the w/cm ratio at the surface and creates a weakened layer of mortar that shrinks and leads to the formation of map cracking, scaling, and other surface defects.
- Avoid sprinkling cement on the surface to dry bleed water: Cement that is sprinkled on the surface in an attempt to absorb bleed water will dry rapidly and shrink, leading to additional map cracking.
- Avoid finishing while bleed water is present. Finishing while bleed water is present on the surface can cause significant weakening of the material at the surface of concrete and contribute to the formation of map cracking and other surface defects.

Use Effective Curing Practices: As described previously, effective curing techniques should be used as soon as possible after concrete placement. This can be accomplished through the use of an approved curing compound at the specified application rate or, when practical, through wet curing methods (fog spraying, wet burlap). Evaporation retarders, which are sprayable solutions that are applied to fresh concrete to reduce the rate of surface moisture evaporation prior to finishing, may also be used.

Use Effective Stockpile Management Practices: Effective stockpile management, from the delivery of the aggregate to its storage and ultimate loading into the mixing plant, will address clay ball issues in concrete. Specific items include cleaning aggregate to remove contaminants prior to stockpiling, minimizing contamination of aggregate stockpiles from the soil below, monitoring stockpile quantities, and avoiding deep stockpile penetrations by the loader operator (ACPA 2004).

An overall summary of causes and prevention is presented in Table 2.3.

Table 2.3 Summary of causes and prevention or mitigation of surface defects

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Map Cracking (Crazing)	Overworking/overfinishing of concrete surface Finishing while bleed water is present on surface Late or inadequate curing (particularly under highly evaporative conditions) Sprinkling cement on surface to dry bleed water Batching absorptive aggregates that are on the dry side of SSD Alkali-aggregate reaction	Design concrete with low permeability Use moderate slump mixtures (with low w/cm ratios)	Use blended cements or SCMs to control AAR Use durable, nonreactive aggregates	Employ proper finishing practices. Do not: • Overwork surface • Spray water on surface during finishing • Finish while bleed water is present • Sprinkle dry cement on surface to dry bleed water Begin curing as soon as possible Use wet curing methods (fog spraying, wet burlap) or evaporation retarders to minimize moisture loss Keep stockpiles wet when absorptive aggregates used	Diamond grinding may be used to remove surface cracking
Plastic Shrinkage Cracking	Rapid evaporation of moisture from the concrete surface	Use durable mixtures with low w/cm ratios Minimize cement contents	Reduce aggregate absorption	Employ proper hot-weather and cold-weather paving practices, as appropriate; if possible, delay the concrete placement until extreme conditions subside Begin curing as soon as possible Use wet curing methods (fog spraying, wet burlap) or evaporation retarders to minimize moisture loss	N/A

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Scaling	Excessive use of deicing salts and freeze-thaw cycles Use of poor finishing and curing practices Failure to protect surface of newly placed fresh concrete from rain Finishing rainwater into concrete surface	Ensure proper air-void system parameters (adequate air content and spacing factor) in concrete		Employ proper finishing practices. Do not: • Overwork surface • Spray water on surface during finishing • Finish while bleed water is present • Sprinkle dry cement on surface to dry bleed water Begin curing as soon as possible Use wet curing methods (fog spraying, wet burlap) or evaporation retarders to minimize moisture loss Protect slab from rain Limit the use of chemical deicers for at least 30 days after concrete placement, if possible	Restore surface texture by diamond grinding
Surface Polishing or Surface Wear	Use of aggregates with poor abrasion resistance Use of improper surface finishing and curing practices Exposure to traffic with studded tires or chains	Use concrete mixtures with adequate strength	Use hard, wear-resistant aggregate	Use proper finishing practices Employ effective curing practices	N/A

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Popouts or Mortar Flaking	Use of unsound or reactive aggregates Erosion of soft, embedded material (e.g., clay balls) Dislodging of mortar above aggregate (flaking)	Use durable mixtures with low w/cm ratios	Avoid the use of problem aggregates Use SCMs or blended cements	Begin curing as soon as possible Use wet curing methods (fog spraying, wet burlap) or evaporation retarders to minimize moisture loss Use proper finishing practices Use effective stockpile management practices to minimize contaminants (ACPA 2004)	N/A

Sources: Modified from Taylor et al. 2007 and PCA 2001

6. Treatment and Repairs

General treatment and repair methods to address concrete pavement surface defects are described below, with an overall summary provided in Table 2.4.

Table 2.4 Treatment options for concrete pavement surface defects

Distresses	Treatment: Penetrating Sealer	Treatment: HMWM	Treatment: Void Filling	Treatment: Diamond Grinding	Treatment: Slab Replacement	Treatment: Overlay
Map Cracking (Crazing) [Due to Construction-Related Factors]		✓ Note: May be applicable if cracks are sufficiently deep		✓ Note: Map cracks generally require no repair but diamond grinding may be used to improve aesthetics		
Map Cracking (Crazing) [Due to AAR Factors]		✓ Has been used to help "glue" distressed pavement together and keep water out to slow the process				✓ Note: Unbonded overlay solutions only, and then only after careful evaluation of extent and severity of AAR, and consideration of future expansion
Plastic Shrinkage Cracking		✓ Note: May be considered for deeper cracks				

Distresses	Treatment: Penetrating Sealer	Treatment: HMWM	Treatment: Void Filling	Treatment: Diamond Grinding	Treatment: Slab Replacement	Treatment: Overlay
Scaling	✓ Note: Use as preventive method prior to salt scaling			✓ Note: Diamond grinding can restore smoothness and improve overall aesthetics	✓ Note: May be appropriate only if extremely severe scaling exists throughout an entire slab	✓ Note: Use when deterioration is too severe and too widespread for any other treatment
Surface Polishing/ Surface Wear				✓ Note: Surface texture improvements may be temporary		✓ Note: Thin asphalt or concrete overlays may be used if structural condition of existing pavement is adequate but these will still be susceptible to studded tires or chains
Popouts/ Mortar Flaking	✓ Note: May help waterproof the surface		✓ Note: Filling of voids larger than 2 in. (51 mm) caused by clay or soil balls and popouts using agency-approved repair materials	✓ Note: Diamond grinding may be used to improve aesthetics		✓ Note: Thin asphalt or concrete overlays may be appropriate when popouts cover a significant portion of slabs within a project

Penetrating Sealers

For surface defects caused by deicing salts, the application of a penetrating sealer (such as silane or siloxane) to the surface of the concrete provides one way of reducing the amount of salt ingress into the concrete. These types of materials penetrate into the concrete to form a chemical barrier that shields against moisture and deicing chemical penetration. Application of these materials at higher concentration levels seems to be more effective than when they are placed at lower concentration levels.

The application of any penetrating sealer should be done only on concrete that is clean and allowed to dry for at least 24 hours at temperatures above 60 °F (15.6 °C); at least 28 days should be allowed to elapse before applying the sealers to new concrete (Kosmatka and Wilson 2016). Penetrating sealers may need to be reapplied after 3 to 5 years.

High Molecular Weight Methacrylate (HMWM)

Low-viscosity HMWM is an adhesive compound consisting of methacrylate monomers. Because of its low viscosity, it achieves excellent penetration into cracks and can serve to strengthen the concrete by filling the crack and bonding it together. These materials are used by some highway agencies to seal fairly deep shrinkage cracks in concrete pavements and bridge decks, and have also been used by some agencies to treat ASR-distressed pavement.

Void Filling

One approach to dealing with clay or soil balls and popouts is to simply clean out and fill the affected area with a repair material (ACPA 2004). This is appropriate for voids more than 2 inches in diameter. Cleaning involves sandblasting and/or high-pressure water, followed by compressed air. Cementitious, epoxy, and proprietary materials have all been used successfully to fill and repair clay ball voids.

Diamond Grinding

Diamond grinding involves the removal of a thin layer of hardened concrete pavement surface using a self-propelled machine outfitted with a series of closely spaced diamond saw blades mounted on a rotating shaft. Diamond grinding is primarily used to restore or improve ride quality but it also improves surface texture and reduces noise. Diamond grinding should be performed after any other appropriate rehabilitation activities (e.g., partial-depth or full-depth patching) but before any surface treatment methods (Smith et al. 2014, Van Dam et al. 2002).

Slab Replacement

Slab replacement involves the complete removal and replacement of the concrete slab, and would likely be appropriate only in the case when the deterioration is extremely severe and present over a large part of the slab (Van Dam et al. 2002).

Overlay

Overlay performance is largely dependent on the type and extent of pre-overlay repairs. Thin asphalt overlays or bonded concrete overlays may be viable treatment options when the existing pavement is in fair to good structural condition, and provided that a materials-related distress (such as AAR) is not the underlying cause of the observed defects. If AAR is confirmed as the problem, an unbonded concrete overlay may be a candidate depending on the extent and severity of the deterioration but its use should be carefully evaluated. Detailed guidance on the selection, design, and construction of concrete overlays is available from the National Concrete Pavement Technology Center (Harrington and Fick 2014).

Where aesthetics is a major concern, thin epoxy overlays or toppings may be an alternative for surface defects not related to materials issues but these systems are typically more expensive.

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CHAPTER 3. SURFACE DELAMINATION

1. Description

Surface delamination (subsequently referred to as delamination) in concrete pavements is closely related in appearance to scaling and spalling. However, the mechanism of failure is different, as will be discussed in detail in this chapter. Delamination may be viewed as the development of a horizontal crack within the slab that results in separation of the surface layer to a depth of 0.5 to 2 inches from the remaining concrete. Delamination may be limited or widespread depending on the basic cause of the separation. Although delamination is normally seen adjacent to pavement joints and may extend 3 feet or more into the slab, as shown in Figure 3.1a, it can also occur anywhere in the slab, as shown in Figure 3.1d.

Figure 3.1 Delamination in concrete pavements

a. Localized delamination in a conventional concrete pavement
Michael Rodwell, Mahaska County, Iowa

b. Close-up view of delamination in the same location is Figure 3.1a
Michael Rodwell, Mahaska County, Iowa

c. Delamination and resultant cracking adjacent to transverse joint
Michael Rodwell, Mahaska County, Iowa

d. Delamination of CRCP

Texas DOT

e. Delamination due to steel mesh located near the surface

Andrew Bennett, Michigan DOT

f. Typical pattern of delamination prior to surface loss
Tom Yu, FHWA

Scaling, as discussed in Chapter 2 of this manual, has a similar appearance to delamination but does not extend beyond approximately 1/2-inch in depth. Compression spalling at joints or transverse cracks may also appear similar but it is due to the intrusion of incompressible materials and is discussed in Chapter 8. Debonding of a bonded concrete on concrete overlay or separation of the lifts in two-lift paving is discussed in Chapters 15 and 16, and therefore is not considered in this discussion of delamination.

2. Severity

Severity levels have not generally been assigned for characterizing surface delamination. However, the extent and depth of the delamination is useful for determining the most feasible rehabilitation option. Therefore, the following severity levels have been developed from the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) with this purpose in mind. The suggested severity levels and method of measurement method used for delamination is summarized in Table 3.1.

Table 3.1 Severity levels and measurement of delamination

Distress	Description	Severity Levels	Measurement
Delamination	Horizontal crack development in a slab that results in delamination (separation) of a relatively thin (approximately 1/2 to 1-1/2 in.) surface layer (12.5 to 37.5 mm) from the remaining substrate concrete layer The thin, delaminated "surface layer" is subject to rapid failure (cracking) under traffic loading The depth of delamination is highly variable depending on the mechanism	Depth less than 1/2 in. (12.5 mm) is classified as scaling **Low**: Average depth from 1/2 to 3/4 in. (12.5 to 19 mm) and/or delaminated area less than 0.5 ft^2 (.045 m) **Moderate:** Average depth from 3/4 to 1 in. (19 to 25 cm) and/or delaminated area ranging from 1/2 to 1 ft^2 (0.45 to 0.9 m^2) **High**: Average depth greater than 1 in. (25 mm) and/or delaminated area greater than 1 ft^2 (.09 m^2)	The delamination should be measured and recorded in terms of the square area (ft^2 or m^2) and corresponding depth in inches The extent of the distress should be carefully mapped to determine if the delamination is localized or widespread Note that the overall surface may not show visible signs of delamination at the time of the survey

3. Testing

In most cases, delamination will show small areas of surface distress before it is detected using standard nondestructive methods. Once suspected delamination is identified in certain areas (which may not be readily apparent by visual inspection unless it is known what to look for), soundings should be taken using chain drag, hammer, or suitable alternatives. These methods can provide the distress limits. Following that, nondestructive tests using mechanical wave technology such as MIRA ultrasonic tomography or multiple impact surface waves (MISW) and/or cores provide the severity details of the delamination.

Refer to Chapter 19 for the appropriate field and laboratory testing protocols required for identifying areas of delamination.

4. Identification of Causes

Delamination can be due to a number of physical and/or materials-related causes, as summarized in Table 3.2. There are numerous theories as to the cause of delamination, none of which is applicable to every situation. Therefore, it is reasonable to assume that a combination of factors is responsible for the development and propagation of delamination in the concrete pavement.

Table 3.2 Summary of physical and material/chemical causes of delamination

Category	Description
Physical	Concrete placement operations can result in a plane of weakness due to differential consolidation or inconsistent delivery of concrete resulting in a horizontal cold joint. Concrete finishing operations can result in overworking the surface (by machine or hand finishing), finishing before bleeding has stopped (particularly for higher slump, hand pours) and excessive evaporation from the concrete surface while waiting to finish and texture. Inadequate or late curing can result in a substantial moisture gradient within the slab potentially leading to high internal stresses (horizontal). Jointing at the proper time, depth, and spacing is necessary to minimize stress build-up within the slab. Shallow or late sawing, particularly in conjunction with long transverse joint spacing results in high internal slab stress development, possibly contributing to delamination. Traffic loading is not thought to be a direct cause of delamination but can exacerbate the problem if a horizontal plane of weakness exists in the slab. Compression shear and subgrade or base/slab friction may be a contributing factor to delamination by providing differential restraint from the top to the bottom of the slab. This, in conjunction with moisture and temperature gradients in the slab, may lead to large horizontal stresses. Continuously reinforced concrete pavements (CRCP) and pavements with embedded steel can delaminate if the steel is placed too close to the surface or is not corrosion resistant.
Material/Chemical	The water to cementitious materials (w/cm) ratio is generally not a problem with machine placed slabs as it is usually 0.38 to 0.42. However, hand pours with a correspondingly higher w/cm ratio can be problematic during finishing operations. The coefficient of thermal expansion (CTE) of the concrete is generally consistent throughout a project. However, temperature differentials throughout the slab can lead to high internal stresses (horizontal) in concrete with a high CTE. The corrosion of embedded steel can result in significant delamination since the steel expands as it corrodes leading to very high internal horizontal stress development at the plane of the steel.

5. Evaluation

Construction-related factors are thought to be a primary cause of delamination as temperature and moisture gradients within the concrete result in high internal stress development soon after concrete placement. Materials-related issues are typically not thought to be a primary cause of delamination. However, they have the potential to exacerbate the development and progression of the delamination.

It is generally accepted that delamination occurs due to a combination of factors, rather than a single factor. The following list provides a basis for evaluation.

Concrete Placement Operations

Cause

Continuous placement (eliminating horizontal cold joints) and uniform consolidation of the concrete is important to ensure uniform strength, minimize segregation, and maintain the proper entrained air content. Failure to achieve any of these can lead to nonuniformity in depth and the potential for a weakened plane in the concrete. It should be noted that the placing equipment must be properly set up, maintained, and inspected. Oil leaks from the hydraulic systems and vibrators will result in a weakened plane in the concrete and subsequent delamination.

Prevention

Concrete should be placed and consolidated using accepted methods that conform to placement specifications and criteria. In general, the concrete should be discharged as close as possible to the final location, avoiding excessive discharge height to minimize segregation, and consolidated sufficiently to remove entrapped air and ensure concrete uniformity with depth.

Concrete Finishing Operations

Cause

Finishing operations generally need to occur after the bleed water in the concrete has evaporated from the surface (Portland Cement Association 2002). However, because of the extensive use of chemical admixtures and supplemental cementitious materials (SCMs) in concrete mixes, bleed water may not readily rise to the surface in a reasonable time or, in some cases, be nonexistent with low w/cm ratio mixes.

Overworking of the concrete surface too soon after placement can result in trapped bleed water at the surface, resulting in a weakened zone that’s subject to delamination. This is much more likely with hand placement but can occur with machine finishing. In addition, the chances for delamination are greatly increased when conditions result in rapid drying of the surface due to evaporation caused by wind, low relative humidity, or direct sunlight. The rapid drying at the slab surface makes it appear that it is ready for finishing when, in fact, the underlying concrete is still in the plastic state and can release bleed water and entrapped air.

Prevention

Hand pour finishing operations should not commence until the bleed water (if present) has evaporated from the concrete surface. Water should not be added to the surface as a finishing aid, and the concrete should not be overworked.

Curing

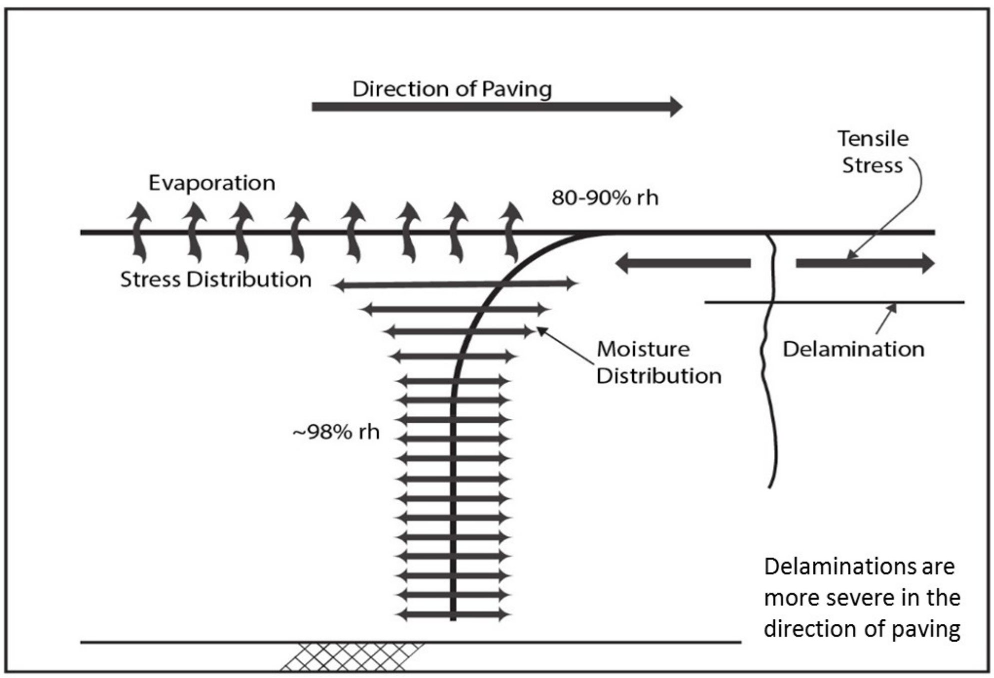
Cause

Curing plays a primary role in strength development and durability of concrete, and must be performed at the proper time using established techniques. Curing supports uniform and adequate moisture in the concrete necessary for hydration and helps to minimize temperature gradients. Rapid surface evaporation leads to a differential moisture gradient within the slab, potentially leading to development of high internal stresses and delamination.

The pavement delamination shown in Figure 3.1 (a, b, and c) is thought to be caused by excessive evaporation. A 2006 report by Mukhopadhyay et al. discusses the effects of surface drying in regards to the cause of delamination. Near surface horizontal shear stresses are developed due to the moisture gradient that exists between the top and bottom of the slab when the surface is allowed to dry due to evaporation. The saw cut is thought to act as an initiation point for the cracking under these conditions.

Figure 3.2 shows the effect of a moisture gradient in delamination of the pavement.

Figure 3.2 Delamination as a result of moisture gradient within the slab



Recreated from Wang and Zollinger 2000

The delaminated areas will typically occur at shallow depths of 1 inch but can be up to 3 inches, and occur at an early concrete age when stresses caused by the moisture variation surpass the concrete shear strength.

Conditions necessary for a delamination to form include low interfacial strength between the coarse aggregate and mortar, and sufficient evaporation of pore water from the hydrating concrete, which results in differential drying shrinkage near the pavement surface. When the delaminated slab experiences curling and warping it can lead to rapid deterioration under traffic loading.

Prevention

Curing should be undertaken as soon as the finishing operations are completed and the water sheen on the surface of the concrete has disappeared. If high evaporation is an issue, the use of an evaporation retarder immediately after placement is recommended. Adequate and timely curing is one of the most critical preventative measures and is dependent on the concrete mix, ambient conditions, pavement structure, and other additional factors. White pigmented curing compound, plastic sheeting, fogging, and other methods of curing also minimize temperature differentials within the slab which are also thought to be a contributor to delamination.

Jointing

Cause

Delamination is oftentimes adjacent to transverse joints and may be due to incompressible materials in the joint or the increased moisture content of the concrete because of moisture intrusion at the joint. Although most compression spalls do not result in delamination of the surrounding concrete, a higher CTE and temperature differential, in conjunction with incompressible materials in the joint, may result in delamination if other factors are present.

Prevention

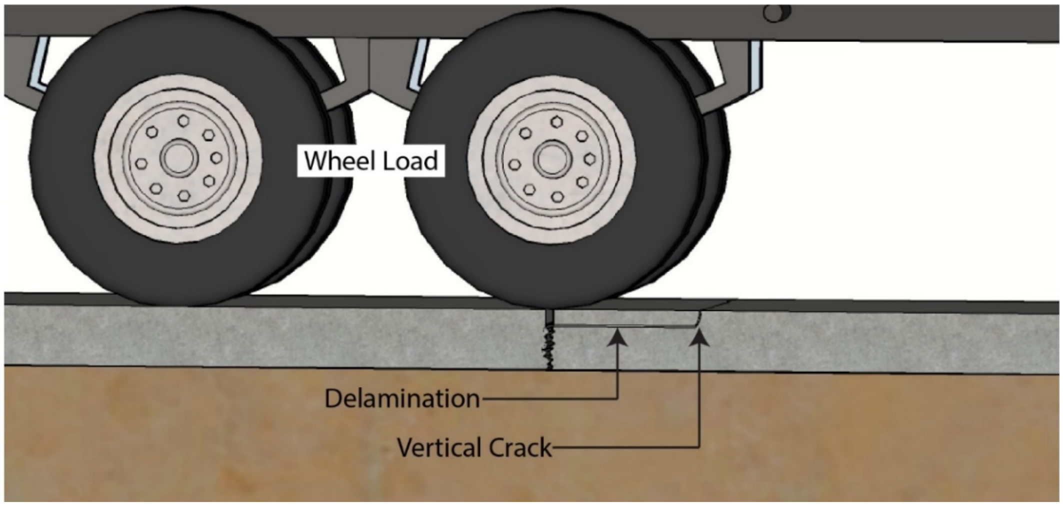
Joint sawing to the proper depth, spacing, and sawing at the optimal time will help to reduce stress build-up within the slab. Adequate joint sealing (if used) can reduce moisture intrusion but more importantly, prevents incompressible materials from entering the joints resulting in contributing to delamination.

Traffic Loading

Cause

Traffic loading leads to slab deflections that may result in delamination if a horizontal plane of weakness already exists in the concrete. Loading only exacerbates a pre-existing condition (such as planes of weakness, poor placement or finishing operations, improper curing, etc.) and would not result in delamination by itself (see Figure 3.3).

Figure 3.3 Delaminated areas leading to increased distress



Snyder & Associates, Inc.

Prevention

Although the formation of delamination planes may not be evident at an the early age, subsequent climatic fluctuations coupled with traffic loading will induce tensile stresses at the area of the delaminated concrete, which will eventually break continuity with the pavement. Depending on the shape and depth of the delamination, trapped moisture, freeze-thaw action, level of stresses, and the fatigue properties of the concrete, shear stress can increase and lead to further delamination, as illustrated in Figure 3.3. To prevent issues caused by traffic loading, it is crucial that traffic does not drive on the concrete prior to the concrete meeting the strength and specifications requirements.

Compression Shear with Weakened Plane

Cause

The relative restraint on the slab imposed by the subgrade or base may contribute to delamination if a plane of horizontal weakness already exists. Delamination from early age compressive stresses developed near the slab surface resulting in aggregate shear can be due to a combination of factors.

Placing the concrete pavement on an existing clay subgrade without a base or subbase can cause delamination. Clay subgrades in Midwestern states are usually at optimum moisture content year-round regardless of weather conditions. This results in nonuniform subgrade drag forces on the new pavement and does not let it slide uniformly, as would happen on the granular base. Thus, the pavement adheres in some areas and slides in others. The top of the pavement is expanding during the day and contracting during the night, resulting in the surface expanding much more in the first hours of the pavement life.

A nonstabilized granular base or subbase has the effect of minimizing the differential drag at the slab or support interface. The internal stress development in the slabs can be attributed to the top surface of the pavement expanding during the day and contracting during the night relative to a greater depth within the slab. Internal stress is particularly problematic soon after placement as the concrete is developing strength but not yet able to withstand the stresses due to the thermal and moisture gradients compounded by the slab or support restraint. The issue is at the top of the slab which experiences no restraint but undergoes movement due to thermal and moisture variations while the bottom of the slab is restrained, resulting in horizontal stress development within the slab.

During the early stages of pavement life, not all of the joints activate (crack under the saw cuts), resulting in high internal stress development due to excessive movement at the activated joints.

These occurrences, coupled with an already unseen weak interface at 3/4- to 1-inch depth due to poor finishing practices, can lead to isolated joint locations of delamination. This weakened plane can grow even weaker with the uplift movement that occurs when the transverse joints are sawn.

Prevention

The use of a granular base/subbase course is highly recommended to enhance pavement performance and to minimize delamination potential caused by nonuniform drag forces.

Steel Placement and Corrosion

Cause

Embedded steel in concrete pavements (i.e., tie bars, dowels, wire mesh, continuous steel, etc.) can result in delamination if placed too near the surface. Figure 3.1e illustrates the effect of “high steel” and the resulting localized delamination. Corrosion of steel placed too near the surface is generally a contributing factor as well. High steel creates a discontinuity within the concrete at an area subject to high stresses due to loading, as well as thermal and moisture gradients within the slab.

Prevention

If embedded steel is specified, corrosion-resistant steel or coated steel are required. Providing adequate depth of cover and low permeability concrete in corrosive environments such as in areas where deicing salts are used is also required.

Water/Cementitious Materials (w/cm) Ratio

Cause

A high w/cm ratio, as would be typical for hand pours, may result in excessive bleed water at the surface of the concrete. Finishing prior to evaporation of the bleed water has the possibility of sealing in the water, thereby creating a weakened plane that can lead to delamination.

Prevention

The w/cm ratio and paste content should be kept as low as practical to minimize bleeding and differential drying shrinkage within the slab. A lower w/cm ratio will also result in higher strength and concrete durability.

Table 3.3 summarizes the common causes and the potential prevention/mitigation strategies for addressing delamination.

Table 3.3 Prevention and mitigation strategies for surface delamination in concrete pavements

Causes	Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Concrete mix-related issues including a high w/cm ratio, paste content, CTE, and reactive aggregates	Use low w/cm ratio mixes where possible Design concrete mixes for low permeability	Use low paste content mixes where possible Do not use high alkali or reactive aggregates if possible; if needed, use SCMs as a mitigation strategy	Adjust concrete workability with admixtures rather than the w/cm ratio	N/A
Poor concrete placement and finishing operations	N/A	N/A	Ensure that segregation does not occur during placement Consolidate the concrete to remove entrapped air but not so much as to remove entrained air or segregate the concrete Do not finish until the surface water sheen dissipates Initiate curing as soon after placement and finishing as possible	N/A

Causes	Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Jointing	Design the transverse joints based on the CTE of the concrete, environment, and subgrade or base/slab friction	N/A	Saw cut joints in a timely manner and to the correct dimensions	Maintain joint sealant to minimize moisture intrusion and incompressible materials from entering joints
Embedded steel	N/A	Use corrosion-resistant steel if in a corrosive environment Use a low permeability concrete mix	Verify that the steel is placed at the correct depth to ensure a sufficient depth of cover	N/A

6. Treatment and Repairs

Pavement delamination can be repaired in a number of different ways depending on the extent and overall condition of the area affected. The following options are viable in the majority of cases but the total extent of the delamination must be assessed prior to selecting a strategy (PCA 2001).

Full-Depth Repairs (Including Slab Replacement)

Full-depth repairs (FDRs) can be used to repair isolated areas of delaminated pavement but becomes cost prohibitive if the area exceeds approximately 10 percent of the pavement. FDR is preferred when the delamination exceeds approximately one-third of the slab depth, and in locations where exposed reinforcing steel is observed (Smith et al. 2014).

Partial-Depth Repairs

Partial-depth repairs (PDRs) can be used to repair isolated areas of delamination where the depth is less than approximately one-third of the slab thickness. It is very important to determine the extent of the delaminated area prior to performing PDR as the repair material must fully bond to the existing sound concrete.

Milling and Diamond Grinding

If the delamination is relatively widespread but shallow, carbide milling followed by diamond grinding (DG) may be an option. The milled surface is not acceptable as a final surface texture but can remove a substantial amount of concrete quickly and at reasonable costs. DG can then be used to provide a smooth ride and a desirable surface texture. Micromilling may also be a feasible repair option and should be evaluated in terms of cost versus anticipated longevity and performance.

Concrete Overlay

A concrete overlay may be the most feasible option if the delamination is widespread. If a bonded concrete overlay is determined to be the most feasible due to grade control issues and other factors, the delaminated concrete will require removal by milling. If an unbonded overlay is determined the most feasible option, pre-overlay repairs will be dictated and may require at least partial milling to remove loosened concrete. The pavement should be re-evaluated after milling to determine if any additional damage is present (Harrington and Fick 2014).

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CHAPTER 4. MATERIAL-RELATED CRACKS

1. Description

Surface Material-related distresses (MRD) are failures that occur in concrete pavements as the result of the properties of the materials in the pavement and their interaction with the environment (Van Dam et al. 2002a). MRDs in concrete pavements are typified by a network of multiple, closely spaced cracks, often accentuated with staining or deposits. However, visual inspection alone cannot confirm the presence or absence of material-related issues. Laboratory testing of pavement core samples is required to definitively confirm the mechanisms that are contributing to the distress.

There are a number of different MRD types that can affect concrete pavement performance, each with unique characteristics and mechanisms. This chapter covers the following MRD types.

- Durability cracking, commonly referred to as D-cracking, is a distress associated with the freezing and thawing of critically saturated, susceptible coarse aggregate particles in the concrete (Van Dam 2002a). Susceptible aggregates fracture and/or dilate as they freeze, resulting in cracking of the surrounding mortar. The distress typically forms in an hourglass shape at transverse joints and cracks and not over the entire slab; furthermore, the deterioration also occurs on the underneath portion of the slab and can actually be more severe at that location because of higher levels of saturation.
- Alkali-aggregate reaction (AAR) describes a family of chemical reactions between certain susceptible aggregates and the alkali hydroxides in the concrete, which can lead to cracking of the concrete matrix. AAR is often a slow process; the deterioration can take several years to develop and progress. There are two forms of AAR: alkali-silica reactivity (ASR) and alkali-carbonate reactivity (ACR).
    - ASR is the most common form of AAR and is a chemical reaction between reactive silica in the aggregate and the alkalis present in the pore solution of the hydrated cementitious paste. The product of the reaction is a gel that expands significantly as it takes in water, destroying the integrity of the weakened aggregate particle and the surrounding cement paste (Van Dam et al. 2002a). ASR can result in map cracking of the concrete or popouts on the surface of the concrete pavement and can also contribute to cracking and spalling. ASR typically forms over the entire slab and not just at joints and cracks.
    - ACR is the reaction between the alkalis in the pore solution of the hydrated cementitious paste and certain carbonate rocks, particularly dolomitic limestones that have a unique texture and the presence of clay. The reaction is highly expansive and can lead to deterioration of the concrete. More detailed information on ACR identification and prevention is available from ASTM International (Ozol 2006), PCA (Farny and Kerkhoff 2007), and FHWA (Thomas et al. 2013).

For the purposes of this chapter, the focus is on the D-cracking and ASR distresses because they are both widely encountered throughout the U.S. and also exhibit prominent cracking features. Should you want more information, a comprehensive three-volume report on the evaluation, identification, and treatment of MRDs is available from the FHWA (Van Dam et. al. 2002a, Van Dam et al. 2002b, Sutter et al. 2002).

Figure 4.1 presents some typical photos of these MRDs. The similarity in some of the characteristics should be noted, which further emphasizes the need for laboratory testing to confirm identification.

Figure 4.1 Various MRDs in concrete pavements

a. D-cracking
Dale Harrington, HCE Services

b. D-cracking
©2018 Applied Pavement Technology, Inc.

c. ACR
Randy Riley, Illinois Chapter, Inc. – American Concrete Paving Association

d. ACR

Randy Riley, Illinois Chapter, Inc. – American Concrete Paving Association

e. ASR

Dan DeGraaf, Michigan Concrete Association

f. ASR
©2018 Applied Pavement Technology, Inc.

2. Severity

The *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) includes D-cracking as a formal distress, and while not specifically mentioning ASR, it does list map cracking, which is an early indicator of ASR distress. However, map cracking can also be caused by a number of other factors (see Chapter 2), so its presence does not necessarily mean that ASR distress is occurring. In many cases, map cracking is the result of highly evaporative conditions or poor finishing practices and does not present any major performance issues.

ASTM International identifies ASR as a distress on standardized pavement condition surveys for airfield pavements (ASTM 2012) but, interestingly, not for roadway pavements. Table 4.1 summarizes the severity levels and measurement methods for D-cracking, map cracking, and ASR based on the available standards. Note that in the definition for ASR, FOD refers to foreign object debris; that is, loose pieces of concrete that pose a particular hazard to aircraft engines on airfield facilities.

Table 4.1 Summary of severity of MRD cracks

Distress	Description	Measurements
D-Cracking **Source: Miller and Bellinger 2014**	Multiple closely spaced crescent-shaped hairline cracks A pattern is typically observed adjacent to cracks or free edges and initiates in slab corners The cracking pattern and its surrounding area is usually discolored	**Low**: Tight cracks with no loose or missing pieces. No patching has been performed in the affected area. **Moderate**: Cracks are well-defined and some small pieces are loose or have been dislodged from the pavement surface. **High**: A well-developed cracking pattern with a significant amount of loose or missing material and pieces up to 1 ft^2 (0.1 m^2) have been dislodged from the pavement surface. The number of slabs with D-cracking and the pavement area affected at each severity level are recorded The slab and the affected area severity rating is based on the highest severity level present for at least 10% of the area affected
Map Cracking/ Pattern Cracking **Source: Miller and Bellinger 2014**	A series of cracks that extend only into the upper surface of the slab Larger cracks are frequently oriented in the longitudinal direction of the pavement and are interconnected by finer transverse or random cracks	No severity levels associated with this distress The number of occurrences and affected area (in ft^2 or m^2) are recorded When an entire section is affected, it is considered as a single occurrence

Distress	Description	Measurements
ASR **Source: Thomas et al. 2011**	The classic symptom of ASR is map cracking (described in the row above), which takes the form of randomly-oriented cracks on the surface of concrete elements that are relatively free (unrestrained) to move in all directions In some cases, discoloration occurs around the cracks, often due to gel exudation in the vicinity of the cracks	**Low**: Minimal to no foreign object debris (FOD) (i.e., loose pieces) from cracks, joints or ASR-related popouts; cracks at the surface are tight (predominantly 1.0 mm or less). Little to no evidence of movement in pavement or surrounding structures or elements. **Moderate**: Some FOD potential; increased sweeping or other FOD removal methods may be required. There may be evidence of slab movement or some damage (or both) to adjacent structures or elements. Medium ASR distress is differentiated from low by having one or more of the following: increased FOD potential, crack density increases, some fragments along cracks or at crack intersections present, surface popouts of concrete, or a pattern of wider cracks (predominantly 1.0 mm or wider) that may be subdivided by tighter cracks. **High**: One or both of the following exist: (1) Loose or missing concrete fragments and poses high FOD potential, (2) Slab surface integrity and function is significantly degraded and pavement requires immediate repairs; may also require repairs to adjacent structures or elements. The number of slabs affected by ASR distress are recorded at the appropriate severity level (ASTM International 2012).

3. Testing

Although the visible features of D-cracking and ASR distress can be strongly suggestive, it is only through additional field and laboratory testing that it is possible for the specific distress mechanisms (and the type of distress) to be confirmed. These additional tests are described below.

More detailed information regarding testing can be found in Chapter 19.

Field Tests

Coring

Concrete core samples can be retrieved for visible inspection to determine the depth of deterioration, the tortuosity of the cracking through the concrete (and either around or through the aggregate), and the presence of any staining or exudate in the cracks. Cores taken away from joints and cracks on D-cracked pavements will generally be in better condition than those taken at the joint or crack.

Field Identification Tests

Several test procedures are available that might be useful in the field identification of pavements suspected of suffering from ASR. These tests use chemicals that are applied to a freshly exposed concrete surface in the field to indicate whether ASR is present. One test uses a uranyl acetate compound that stains the products of the ASR to make them visible under ultraviolet light, whereas another uses a nonradioactive chemical (sodium cobaltinitrite) to stain the ASR products. However, these tests have seen limited use for the field identification of ASR, and neither should be considered as definitive in the identification of ASR (Farny and Kerkhoff 2007).

In addition, signs of expansion within a project—such as joint sealant compression failures, spalling, blowups, or shoving of fixed structures—are possible indicators of ASR.

Laboratory Tests

As mentioned previously, laboratory testing is essential for the identification and confirmation of MRD. This testing is normally performed on samples from the field coring program, and should consist of petrographic examination (performed in accordance with ASTM C856) and possibly additional chemical and mechanical testing. As part of this evaluation, relevant project documentation (e.g., design and construction reports, mix design information, materials testing reports, etc.) should be gathered and reviewed. Common aggregate testing includes ASTM C666/AASHTO T 161 for D-cracking and ASTM C1260, ASTM C1293, or ASTM 1567 for ASR.

More detailed information regarding field and laboratory testing can be found in Chapter 19.

4. Identification of Causes

Table 4.2 provides descriptions of the physical and material/chemical factors that are related to the development of these two primary MRDs.

Table 4.2 Physical and material/chemical causes of MRDs

Distress	Category	Description
Durability Cracking (Physical)	Freezing and Thawing	Freezing and thawing of critically saturated, susceptible coarse aggregates that result in fracturing and/or damaging dilation of the aggregate
Alkali Aggregate Reaction (Material or Chemical)	Deleterious Expansion	Chemical reaction between the alkalis in the cement paste and certain components in aggregates that result in deleterious expansion.

5. Evaluation

The following are detailed descriptions of the various causes (or distress mechanisms) of the two primary MRD types and methods for preventing or controlling them.

D-Cracking

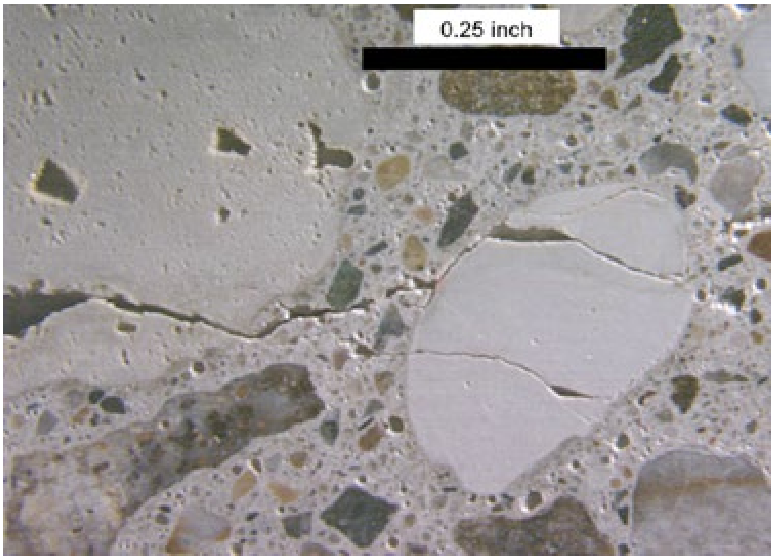
As described previously, D-cracking is the deterioration of a critically saturated, susceptible coarse aggregate in a concrete mixture caused by freeze-thaw cycles. D-cracking is predominantly located in the Midwestern parts of the U.S. (e.g., Kansas, Missouri, Illinois, Indiana, Iowa, Minnesota, Wisconsin, Michigan, Ohio, and other neighboring states) where susceptible aggregates such as limestone, dolomitic rock, and chert are common. Susceptible aggregates generally have higher total porosity and a higher proportion of medium-sized pores (0.1 to 5 μm), which allows for saturation of a significant volume of water in freezable pore space (Kosmatka and Wilson 2016).

D-cracking initially appears as a series of fine cracks generally running parallel to joints, cracks, or free edges, all exposed locations where moisture can easily intrude. It also can develop at the bottom of the slab where excess moisture can accumulate, although the depth of freeze-thaw cycling may affect this (Janssen and Snyder 1994). D-cracking may not appear for 15 or more years after construction but this varies depending on the characteristics of the aggregate.

Cause

Three factors are needed for D-cracking to develop: 1) the concrete contains aggregates susceptible to D-cracking in sufficient quantity and size, 2) the concrete is exposed to sufficient moisture, and 3) the concrete is exposed to repeated cycles of freezing and thawing (Janssen and Snyder 1994). The process begins with water filling the pores in the susceptible aggregates, which then undergoes cycles of freezing and thawing during the cold winter months. Since the pore structure of the aggregates does not facilitate the easy escape of water, the expansion due to the phase change from liquid water to ice results in the development of internal stresses and under repeated freezing and thawing cycles, the aggregates eventually fracture (see Figure 4.2) or dilate, thus damaging the surrounding mortar.

Figure 4.2 Fractured aggregate in D-cracked pavement



Karl Peterson, Michigan Tech

The distress typically manifests in the form of crescent-shaped cracks parallel to joint or cracks on the surface of the concrete pavement. A dark staining due to calcium hydroxide or calcium carbonate residue accompanies the cracking, often in an hourglass shape on the pavement surface at affected joints and cracks (see Figure 4.3).

Figure 4.3 Staining accompanying D-cracking at joints

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Prevention

The most obvious and effective way of preventing D-cracking is to avoid the use of susceptible aggregates. In the last several decades, highway agencies have made significant strides in identifying their problem aggregate sources but an effective aggregate testing program is still needed to screen for susceptibility to freeze-thaw deterioration. The most common screening test being used is AASHTO T 161/ASTM C666, which at the end of the test yields a durability factor (DF) and a length change. Acceptance threshold values for these factors are set by each agency, and can vary depending on the climate, exposure conditions, and aggregate type (Kosmatka and Wilson 2016).

Aggregate susceptibility to freeze-thaw deterioration is generally reduced as particle size is reduced (Van Dam 2016). Although the critical size at which an aggregate will fail is dependent on many factors, many highway agencies have had good success in minimizing D-cracking by reducing the maximum size of susceptible coarse aggregate to 0.75 inches or less. This is effective in reducing freeze-thaw deterioration but the use of smaller top size coarse aggregate will result in increased paste content and may affect the volume stability of the concrete and may also reduce aggregate interlock at joints and cracks.

It should be noted that the air entrainment of the concrete mixture does not prevent the development of D-cracking. That entrained air is placed to protect the paste and will have no effect on the freeze-thaw deterioration of the coarse aggregate.

Alkali-Silica Reactivity (ASR)

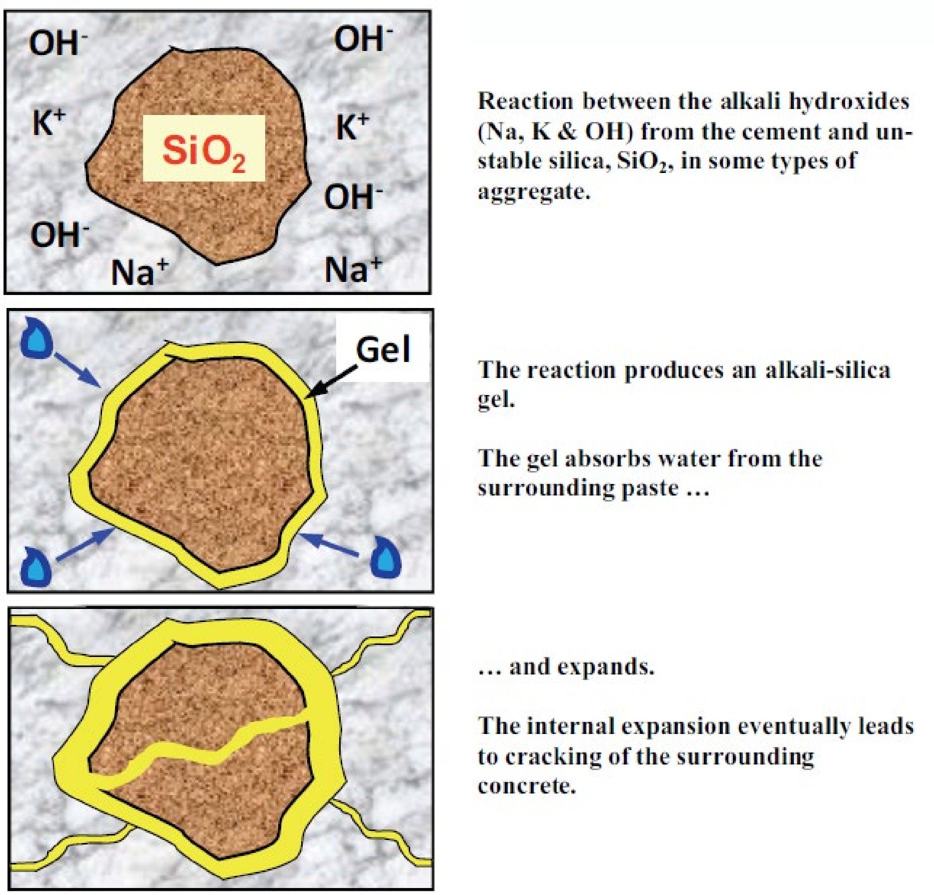
As defined previously, ASR is a deleterious reaction between alkalis in the pore solution and reactive silica in aggregate. The product of the reaction is an expansive gel that leads to cracking in the concrete matrix. Although at one time thought to be limited to the western parts of the U.S., ASR is now recognized as being widespread throughout North America (Thomas et al. 2011). Common susceptible aggregates include chert, quartzite, gneiss, and shale, among others.

ASR results in the development of a map cracking pattern on the pavement surface, starting at the joints and then working across the entire slab. Upon continued development, the cracks can begin to spall and deteriorate, and the pavement may also exhibit blowups, shoving, and other pressure-related damage (Van Dam 2002a). In some cases, a gel exudate may appear in the cracks. ASR may appear as early as 5 years after construction, although 10 to 15 years may be more typical when susceptible aggregates are used and no mitigation methods are employed.

Cause

There are three requirements for deleterious ASR to occur, as described below and illustrated in Figure 4.4.

Figure 4.4 Sequence of ASR development



Thomas et al. 2011

For ASR to occur, the concrete must have the following:

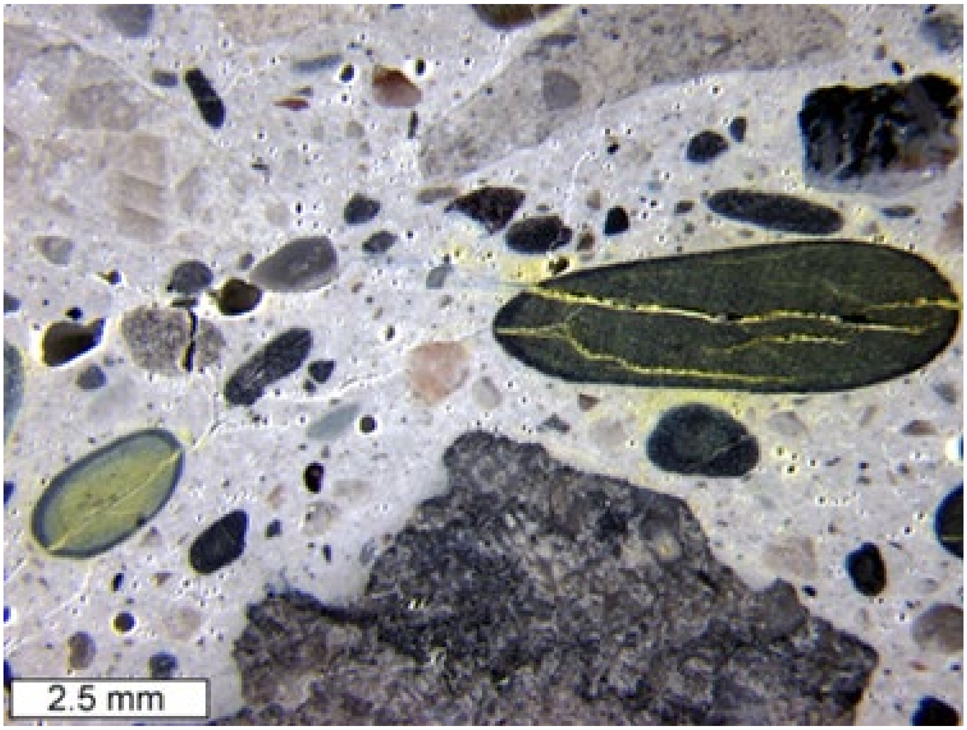
- A sufficient concentration of alkali hydroxides (sodium hydroxide, NaOH, and potassium hydroxide, KOH) in the pore solution of the concrete. The main source of alkalis in concrete is the portland cement but additional alkalis may come from other components of the concrete (e.g., aggregates, admixtures) or from external sources (e.g., deicing salts, seawater).
- A sufficient quantity of unstable silica in the aggregate.
- A sufficient supply of moisture in the concrete. The ASR reaction ceases below a relative humidity of 80 percent but increases in intensity as the relative humidity within the concrete increases from 80 to 100 percent.

The elimination of any one of these requirements will prevent the occurrence of damaging ASR.

As explained by Thomas et al. (2013), ASR is initiated by a reaction between the hydroxyl ions in the pore solution and certain types of silica in the aggregate. In the highly alkaline environment, certain forms of silica tend to dissolve out, making them available to react with the alkalis. The result is an alkali-silicate gel that absorbs water and swells if in a moist environment, leading to the fracture of affected aggregate particles (see

Figure 4.5) as well as the surrounding concrete matrix. The gel that is produced through the reaction appears as a glassy-clear or white powdery deposit within reacted aggregate particles (Van Dam et al. 2002a) and may not always be visible to the naked eye.

Figure 4.5 Aggregate particle fracturing due to ASR



Karl Peterson, Michigan Tech

Prevention

There are four strategies available to mitigate ASR (Thomas et al. 2013) (which are described further below):

- Avoid the use of susceptible aggregates
- Use supplementary cementitious materials (SCMs)
- Minimize the total alkalis in the concrete mixture, considering both the alkalinity and amounts of the cementitious materials contained in the mixture
- Use lithium-based admixtures

Avoid the use of susceptible aggregates. Aggregate reactivity can be assessed using a thorough testing program, including a combination of petrographic analysis (ASTM C295), expansion testing of mortar (AASHTO T 303/ASTM C1260), or concrete (ASTM C1293), coupled with an evaluation of field performance. However, it is emphasized that this option may not always be the best, as nonreactive aggregates might not be locally available, local reactive aggregates with otherwise suitable properties for use in concrete are available at low cost, or because of uncertainty in the test results.

Use supplementary cementitious materials (SCMs). When contributing a small amount of alkalis, the addition of SCMs is effective because it will combine with alkali hydroxides in the concrete and produce additional calcium silicate hydrates, thereby reducing the alkali hydroxides in the concrete (Taylor 2015). Class F fly ash is generally considered most effective, and Table 4.3 provides general recommendations regarding typical SCM levels, expressed as a percentage of the total cementitious materials.

Table 4.3 Required levels of SCMs to control ASR

Type of SCM	Level Required (as % of Total Cementitious Materials)
Low-calcium fly ash (Less than 8% CaO; typically Class F fly ash)	20 to 30
Moderate-calcium fly ash (8 to 20% CaO; can be Class F or Class C fly ash)	25 to 35
High-calcium fly ash (Greater than 20% CaO; typically Class C fly ash)	40 to 60
Silica fume	8 to 15
Slag cement	35 to 65
Metakaolin (Calcined kaolin clay)	10 to 20

Source: Thomas et al. 2013

It is emphasized that SCMs have varying degrees of effectiveness and should be thoroughly tested to ensure their impacts on controlling ASR. ASTM C1567 can be used to evaluate the effectiveness of SCMs in mitigating or controlling ASR.

Minimize the total alkalis in the concrete mixture, considering both the alkalinity and amounts of the cementitious materials contained in the mixture. Although many specifications allow the use of potentially reactive aggregates provided that the cement alkali content does not exceed 0.6 percent sodium oxide equivalent (Na_2Oe), it is now recognized that it is better to consider the total alkalis in the concrete mixture. The required limit will vary with a number of factors but a maximum alkali limit of 3.0 lbs./yd^3 Na_2Oe has been adopted in the AASHTO Provisional Protocol (PP) R 80 for a high level of prevention.

Use lithium-based admixtures. Lithium compounds, particularly lithium nitrate, have been shown to be effective in mitigating ASR but their effectiveness is highly dependent upon the type of aggregate. Thus, it is not possible to prescribe a universal dosage level of lithium to control ASR with the required levels determined by testing various amounts of lithium with the specific aggregate being considered for use.

AASHTO PP R 80, Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction, provides guidance to highway agencies for the determination of the reactivity of concrete aggregates and the selection of preventive measures, using either a performance or prescriptive based approach. Aggregate reactivity for ASR is judged in AASHTO PP 65 (now PP R 80) as follows (Van Dam 2016):

- Consideration of field performance, taking into account any differences in materials and mixture design that may have occurred.
- Petrographic analysis of the aggregate (ASTM C295) to determine the presence of potentially reactive minerals.

- Mortar bar expansion (AASHTO T 303/ASTM C1260) not greater than 0.10 percent after 14 days immersion in a 1M NaOH solution at 176 °F, and expansion of concrete prisms (ASTM C1293) at not greater than 0.040 percent at 1 year. Note that a test period of 2 years is required for ASTM C1293 if the effectiveness of SCMs is being investigated.

If the aggregate is identified as being potentially ASR reactive, AASHTO PP R 80 requires that it be rejected or used with appropriate preventive measures based on the level of ASR risk and the classification of the structure.

It should be noted that the AASHTO T 303/ASTM C1260 test is known to produce some erroneous results when compared to the ASTM C1293 test. When results from both tests are available, the ASTM C1293 data should be relied upon for assessing aggregate reactivity (Thomas et al. 2013).

Table 4.4 summarizes the key points related to the causes and prevention of D-cracking or ASR distress in concrete pavements. Many of the preventive maintenance procedures do not directly mitigate the deterioration but may be effective as short-term measures to maintain the serviceability of the roadway until more significant actions can be implemented.

Table 4.4 Summary of causes and prevention of D-cracking and ASR distress in concrete pavements

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
D-Cracking	Use of coarse aggregates that break down and/or dilate under repeated freeze-thaw cycles when critically saturated	Consider the use of effective joint seals and provision of subsurface drainage to try to reduce moisture in pavement	Certified aggregate sources Avoid using at-risk aggregates Limit the maximum size of coarse aggregates	Certified aggregate sources Consider the use of effective joint seals and provision of subsurface drainage Inspect subdrain systems at the time of construction to ensure their functionality	Keep joints well sealed and maintain drainage systems.
ASR Distress	Chemical reaction between certain siliceous constituents in susceptible aggregates and alkali hydroxides in cement paste that can lead to expansion and cracking	Consider the use of effective joint seals and provision of subsurface drainage to try to reduce moisture in pavement	Certified aggregate sources Use durable, nonreactive aggregates Reduce alkali content of cementitious system Use appropriate type and amounts of SCMs Select material by following guidelines specified in AASHTO PP 65 (now PP R 80) Consider use of lithium-based compounds in mixture	Certified aggregate sources Consider the use of effective joint seals and provision of subsurface drainage Inspect subdrain systems at the time of construction to ensure their functionality	Topical applications of surface sealers

Sources: Van Dam et al. 2002b, Thomas et al. 2013, Taylor 2015, Van Dam 2016

6. Treatment and Repairs

General repair methods and maintenance approaches to address D-cracking and ASR distress are described in the following sections.

Repairs

Partial-Depth Repair (PDR)

PDRs are one rehabilitation method that can be used to repair localized deteriorated areas. These repairs consist of the removal of concrete near the surface and replacement with an acceptable patch material, often a rapid-setting material to limit closure time. However, their effectiveness is limited to smaller areas where the deterioration is confined to the upper one-third to one-half of the concrete slab. Furthermore, PDRs are not always an ideal repair approach because of the progressive nature of the distress in which deterioration can continue to develop beyond the boundaries of the patch (Van Dam 2002a). Still, they may serve as effective short-term solutions until more substantial rehabilitation can be programmed but their cost-effectiveness should be evaluated in comparison with other stopgap measures. More information on PDRs is available in the *Concrete Pavement Preservation Guide* available from the National Concrete Pavement Technology Center (Smith et al. 2014).

Full-Depth Repair (FDR) and Slab Replacement

FDR and slab replacement are commonly used to address D-cracking and ASR distresses in concrete pavement. These repairs consist of the removal of isolated deteriorated areas through the entire thickness of the existing slab and the replacement with a concrete replacement material. Coring can determine the extent of deterioration and the size of patch that would be required. As with all repair methods, FDRs should be viewed not as a solution to the durability issue but rather as a means to extend the life of the pavement until more substantial rehabilitation can be performed (Van Dam et al. 2002a). Furthermore, it likely will not be cost-effective if repairs are required for most every joint on a particular project or if very short performance periods are being considered. The *Concrete Pavement Preservation Guide* (Smith et al. 2014) provides details on the design and construction of effective FDRs and slab replacements.

Retrofitted Edge Drains

The addition of retrofitted drains is intended to remove moisture from under the slab and at joints and cracks, which may assist in slowing or delaying some MRD development. However, the effectiveness of retrofitted edge drains is largely dependent on the characteristics of the base materials, which may limit their effectiveness, particularly for very aggressive forms of MRD. Furthermore, the presence of the edge drains does nothing to address the deterioration that has already occurred in the pavement. More details on retrofitted edge drains is provided by Smith et al. (2014).

Unbonded Concrete Overlay

Unbonded concrete overlays can be effective rehabilitation treatments for existing pavements affected by MRD since their performance is less dependent on the condition of the underlying pavement. These are essentially designed as new concrete pavements, and the need for removal and disposal of the existing pavement is eliminated. However, severe cases of MRD might lead to poor support conditions and significant expansive forces that might limit the appropriateness of an unbonded overlay. Additional information is available in the National Concrete Pavement Technology Center's *Guide to Concrete Overlays* (Harrington and Fick 2014).

Maintenance

Joint Filling Sealing

Because moisture is a contributing factor to the development of many durability-related distress types, the reduction in the amount of water that infiltrates a pavement system may help reduce the progress of MRD development. In an existing pavement structure, filling the joints and cracks helps to minimize water infiltration and may have some marginal impacts but it should be recognized that moisture can still infiltrate the pavement from other locations. In addition, the overall effectiveness of the joint and crack filling can vary depending on a number of factors including climate, joint design, joint filler material, and provision of subsurface drainage, among others. At best, this may be a short-term solution to minimizing water infiltration and does nothing directly to address any deterioration that has already occurred.

Edge Drain Maintenance

If an existing pavement has a drainage system, its ongoing maintenance is essential to ensure its overall effectiveness and functionality. At a minimum, this includes periodic inspections (both of the outlets and of the internal drainage system itself) and may also require cleaning and flushing of the system.

Topical Treatments

A variety of topical treatments may be applied to the surface of concrete pavements in an effort to mitigate or delay the development of MRDs. These include the following (Sutter et al. 2008, Thomas et al. 2013):

- Surface sealers (e.g., silanes, siloxanes) that are used to reduce or prevent the ingress of moisture (as well as other substances such as deicing chemicals) into the pavement structure. Higher concentration levels may be more effective than lower concentration levels, and these would require future re-application for long-term effectiveness.
- Lithium compounds, used to reduce ASR expansion in pavements already suffering from ASR. Lithium nitrate is most commonly used for this application.
- High-molecular weight methacrylate, used to “glue” a deteriorated concrete pavement together and buy time before more significant rehabilitation can be programmed.

One issue associated with all of these materials is their depth of penetration, since as a topical application they predominantly treat only the surface. Consequently, their effectiveness can be limited.

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CHAPTER 5. TRANSVERSE AND DIAGONAL CRACKING

1. Description

Cracking refers to a distinct fracture in a jointed plain concrete pavement (JPCP). Transverse cracking, also called mid-panel or mid-slab cracking, is oriented laterally across the pavement and perpendicular to the pavement centerline whereas diagonal cracking is oriented obliquely across a slab, at roughly a 30- to 60-degree angle from the pavement centerline. Slab cracking may also develop longitudinally, which means that the crack is oriented parallel to the pavement centerline. This chapter focuses on transverse and diagonal cracking; Chapter 6 provides detailed information on longitudinal cracking.

Regardless of orientation, these types of cracks are differentiated from map cracking or other surficial cracking (see Chapter 2) in that they are distinct cracks that typically extend through the entire thickness of the slab. Moreover, these cracks can also develop in conjunction with one another to produce what is often referred to as a shattered or broken slab (in which the slab is divided into three or more pieces). Although cracking is perhaps the most common structural distress in concrete pavements, not all cracks are necessarily indicative of structural failures. Figure 5.1 presents some typical photos of transverse and diagonal cracking.

Figure 5.1 Transverse and diagonal cracking in concrete pavements

a. Transverse crack on JPCP
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b. Multiple transverse cracks on JPCP

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c. Diagonal crack on JPCP

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d. Shattered or broken panel
Kim Willoughby, Washington State DOT

2. Severity

The *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) lists transverse cracking as a distress for JPCPs. Table 5.1 summarizes the severity levels and measurement methods for transverse cracking. Note that while diagonal cracking is not explicitly called out in the *Distress Identification Manual for the Long-Term Pavement Performance Program*, its measurement and severity levels are considered to be the same as those shown for transverse cracking. Furthermore, it is noted that diagonal cracking is generally differentiated from corner breaks as cracks that intersect the joints at distances greater than half of the slab length or width (ASTM 2016).

Table 5.1 Severity levels and measurement

Distress	Description	Severity	Measurement
JPCP: Transverse Cracking*	Cracks that are predominantly perpendicular to the pavement centerline	**Low**: Crack widths less than 0.125 in. (3 mm), no spalling and no measurable faulting or well-sealed and the width cannot be determined **Medium**: Crack widths greater than 0.125 in. (3 mm) but less than 0.25 in. (6 mm) or with spalling less than 3 in. (75 mm) or faulting up to 0.25 in. (6 mm) **High**: Crack widths greater than 0.25 in. (6 mm) or with spalling greater than 3 in. (75 mm) or faulting greater than 0.25 in. (6 mm)	Record the number and length of transverse cracks at each severity level. Rate the entire transverse crack at the highest severity level present for at least 10% of the total length of the crack. Also record the length of the transverse cracking at each severity level with sealant in good condition. The total length of the well-sealed crack is assigned to the severity level of the crack. Record only when the sealant is in good condition for at least 90% of the length of the crack. When a crack is within 1 ft (0.3 m) of a joint for only a portion of its length, it should be recorded as a spall only for that portion so long as that portion is at least 1 ft (0.3 m) long. The portion of the crack that is greater than 1 ft (0.3 m) from the joint should be recorded as a transverse crack.

* Also applicable to diagonal cracking

Source: Modified from Miller and Bellinger 2014

For JPCP performance modeling in the current mechanistic-empirical design methodology, the cracking performance measure employed is the percentage of slabs exhibiting transverse cracking (AASHTO 2015). These cracks should extend at least one-third of the slab width for counting purposes.

3. Testing

Field and laboratory tests are described below. More detailed information can be found in Chapter 19.

Field Tests

Following the pavement distress survey in which transverse and diagonal cracking are identified and recorded, some follow-up field testing may be needed to help determine possible causes of the distress, as well as to assist in the identification of appropriate preservation or rehabilitation solutions. These additional field tests include the following.

Coring

Concrete core samples can be retrieved at the cracks to determine the depth of the crack penetration through the slab, as well as to verify the thickness of the slab. This also provides the opportunity to observe the tortuosity of the cracking through the concrete (and either around or through the aggregate) and the condition (and potential frictional characteristics) of the underlying base/subbase below the crack. Cracking around an aggregate is suggestive of an early cracking mechanism while cracking through the aggregate is suggestive of structural cracking but these guidelines are not absolute (Walker et al. 2006).

Cores of adjacent transverse joints can also be retrieved to measure the depth of the joint sawing (for comparison with specification requirements) and to determine if cracks have activated below the weakened plane; these cores may be more appropriate on projects exhibiting early-age cracking, with coring of transverse joints appropriate when early-age transverse and diagonal cracking have occurred. If present, any steel reinforcement in the slab may be inspected for signs of corrosion or deterioration.

Falling Weight Deflectometer (FWD) Testing

Deflection testing using the FWD can be performed to evaluate the underlying support conditions to determine if that may have contributed to the development of the cracking. In addition, the FWD can be used to assess the load transfer characteristics of the transverse or diagonal cracks and to detect the presence of voids beneath the slab.

Laboratory Tests

Laboratory testing may be conducted if there are concerns about the volume characteristics of the slab potentially contributing to the slab cracking. This could include evaluation of the coefficient of thermal expansion (CTE), as well as potential petrographic analyses to provide insight on mix proportioning parameters (e.g., w/cm, cement content, SCM content, air void system characteristics, etc.).

4. Identification of Causes

Concrete pavements crack whenever critical stresses in the slab exceed the tensile strength of the concrete. To minimize the potential for slab cracking, a system of joints (both transverse and longitudinal) are introduced into the pavement to promote the development of the cracks at predetermined locations where they can be controlled, maintained, and furnished with dowel bars for positive load transfer. This helps minimize the magnitude of excessive stresses that could lead to early-age cracking in the young concrete, while long-term fatigue cracking is assumed to be controlled by an effective slab thickness design for the prevailing conditions. Table 5.2 provides a summary of the factors that can contribute to the development of transverse and diagonal cracking in concrete pavements.

Table 5.2 Summary of physical and material/chemical causes of transverse and diagonal cracking

Distress	Category	Description of Causes
Transverse and Diagonal Cracking	Physical	Environmental conditions (e.g., temperature, humidity, joint lockup) Slab characteristics (e.g., improper joint spacing/layout, inadequate thickness) Construction-related aspects (e.g., timing/depth of joint sawing, curing, restraint cracks from adjacent lanes, utilities) Traffic loading (e.g., age at loading, truck applications, traffic channelization)
Transverse and Diagonal Cracking	Material/Chemical	Concrete mixture properties (e.g., mix proportioning, strength development, shrinkage, CTE) Foundation support conditions (e.g., stiffness, density, bonding/frictional conditions, erodibility, instability, variability, swelling, frost heave)

5. Evaluation

Transverse and diagonal cracking in JPCPs generally develop as the result of a combination of several of the factors listed above. The timing of the crack appearance can vary significantly, with some cracking potentially occurring very soon after construction (e.g., transverse cracking caused by the late sawing or omission of a jointing system), while other cracking may take several years to develop (e.g., transverse fatigue cracking caused by repeated truck applications).

A particular concern to roadway agencies is the development of early-age cracking, which is commonly taken as cracking that occurs within the first 72 hours after paving. Shortly after concrete is placed, it undergoes volume changes as the result of moisture losses (hydration and evaporation) and temperature changes. In addition, differences in moisture levels and temperatures between the top and bottom of the slab may exist, which elicit curling (due to temperature differentials) and warping (due to moisture differentials) responses (see Chapter 10 for more information on curling and warping).

If these volume changes and curling/warping responses are restrained (for example, by the frictional forces between the slab and base), and timely efforts are not made to alleviate those responses through effective joint sawing or formation, random cracking can develop in the young concrete if its tensile strength (which is dependent upon mix parameters, environmental conditions, and curing regimes) is exceeded. These early-age, random cracks have the potential to compromise the structural and functional performance of the pavement and can present ongoing maintenance issues.

Analytical software (e.g., HIPERPAV, www.hiperpav.com) is available to help assess the potential for early-age cracking for a set of concrete mix parameters, design inputs, and paving conditions (Ruiz et al. 2015). More detailed information on early-age cracking is available from several resource documents (ACPA 2009, Taylor et al. 2006, and National Concrete Pavement Technology Center 2007).

This section describes various types of transverse and diagonal cracking—both early-age and long-term—that are attributable to specific causes. Preventive measures for minimizing the development the specific type of cracking is also presented but it is again emphasized that cracking in concrete slabs often develops as the result of several factors working together.

Transverse Cracking Due to Volumetric Changes

Transverse cracking (see Figure 5.2) can develop on concrete pavements as the result of volume changes that occur in the slab and typically extend through the entire thickness of the slab.

Figure 5.2 Transverse cracking caused by volumetric changes

Dan DeGraaf, Michigan Concrete Association

Cause

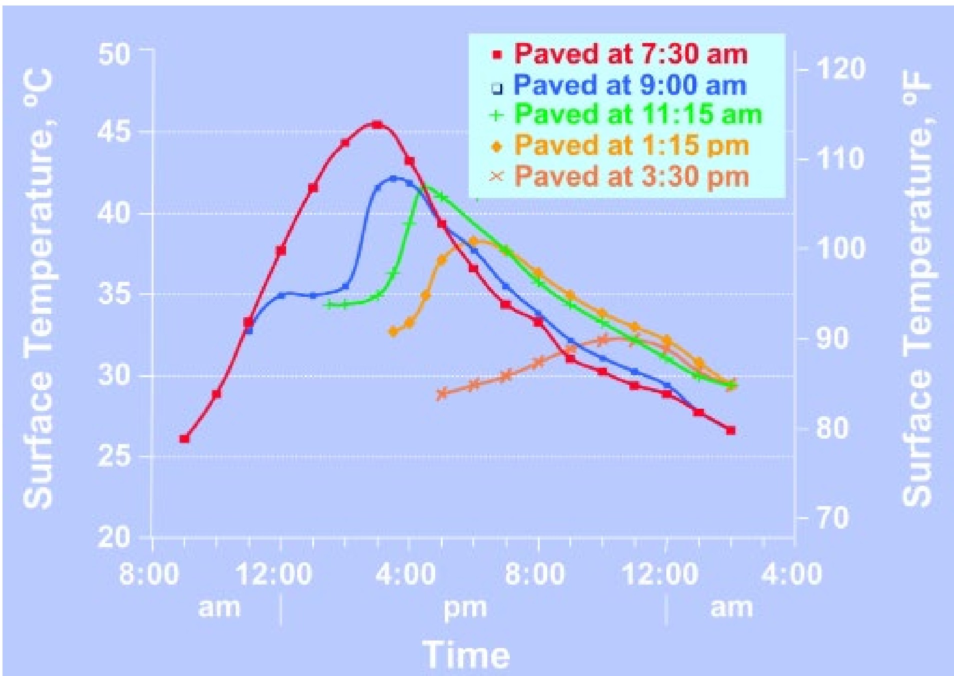
Volumetric changes. Cracking caused by volumetric changes can be due to both concrete mixture properties and environmental effects, and as a result may be subgrouped as either drying shrinkage cracking or thermal contraction cracking. Drying shrinkage cracking is related to the loss of moisture from the slab and occurs after the slab has reached a hardened state, often taking several weeks or months to appear (Neville 1996, TRB 2006). Among the concrete mixture properties and characteristics that are associated with increased risk for drying shrinkage cracking are a high w/cm ratio, high cement contents, high early strength cements, and high CTE values of the course aggregate (Taylor et al. 2006).

Thermal contraction. Thermal contraction cracking is related to the environmental conditions at the time of construction. Generally speaking, environmental factors that affect the curing and strength development of a concrete slab (temperature, humidity, wind, direct sunlight) also influence the potential for early-age thermal contraction cracking. Temperatures can play a critical role, such that elevated ambient temperatures can induce an early set to the concrete that leads to significant contraction (and potentially cracking) later when it cools. Lower ambient temperatures can affect the rate of strength gain of the concrete, potentially making it more susceptible to early-age cracking. Thermal contraction cracking may occur anywhere from one day to several weeks after placement (Neville 1996, TRB 2006).

Ambient temperatures. A particular concern at the time of concrete pavement construction is any sudden change in ambient temperatures. For example, a quick-moving storm front may rapidly cool a recently placed concrete pavement, establishing a higher temperature gradient through the slab and increasing the possibility for early-age cracking. Even significant temperature drops overnight (as may be commonly experienced during spring and fall paving events) may produce significant temperature gradients that could lead to early-age transverse cracking.

Another important consideration related to temperatures and environmental conditions is the time of day that the concrete is placed. As shown in Figure 5.3, concrete that is placed earlier on a typical summertime day of paving reaches a higher peak temperature than concrete that is placed later in the day.

Figure 5.3 Concrete pavement surface temperatures associated with different paving times



Shiraz Tayabji, Applied Research Associates

This happens because the peak heat of hydration of the concrete (which typically occurs about 4 to 8 hours after paving) coincides with the hottest part of the summer day, leading to elevated temperatures in the concrete and a greater risk for early-age cracking (Taylor et al. 2006). Avoiding the early morning hours for paving or making suitable adjustments to the concrete mix are possible approaches to reducing the elevated peak temperatures that could be experienced during summertime paving.

Prevention

The development of concrete mixtures with reduced shrinkage characteristics is important in minimizing the development of drying shrinkage cracking. Table 5.3 summarizes some of the recommended concrete mixture modifications.

Table 5.3 Concrete mixture modifications to help control drying shrinkage cracking

Method of Control or Prevention	Why This Works
Avoid using high early strength cements (Type III cements) unless for special conditions	High early strength cements increase shrinkage potential by generating heat at a faster rate
Use aggregate with low CTE	Minimizes aggregate expansion to help control volume change
When possible, avoid high paste content	High paste and, thus, high water content increase shrinkage potential
Consider using a water-reducing admixture	Reduces paste content and helps reduce shrinkage
In hot weather, consider using set- retarding admixture	Reduces heat generated, thus reducing thermal contraction
Incorporate fly ash or slag cement	Reduces thermal shrinkage; in comparison to portland cement, these materials lower the amount of heat generated while extending the hydration process
During hot weather, consider using pre-cooled materials in the batch (e.g., shade and dampen aggregates, use chilled water/ice in the mix)	Lowers the temperature of the mix and, thus, minimizes the amount of cooling and shrinkage after final set
During cool weather, use warm or heated water in the mix	The warmer water helps to initiate and maintain an adequate hydration process
Keep aggregate at a moisture state of saturated surface dry or greater and tightly monitor moisture content	Dry aggregates absorb moisture meant for cement hydration; saturated aggregates add water to the mix, which can increase the w/cm ratio and reduce durability
Use well-graded aggregates	Requires less paste and thereby less water, which leads to a lesser potential for shrinkage

Source: Modified from National Concrete Pavement Technology Center 2007

Although environmental factors are generally out of the control of the paving contractor, recognizing the potential impacts of environmental conditions on early-age thermal cracking development is the first step in minimizing their effects. If the environmental conditions become severe, the contractor may need to suspend paving operations or adjust the time of day when concrete is placed, or perhaps make suitable adjustments to the concrete paving mix in order to avoid or minimize environmental impacts.

Recommendations for concrete paving under extreme hot and cold weather conditions, respectively, are provided below.

Hot weather conditions are generally taken as a combination of elevated temperatures, low humidity, and high wind speed that can adversely impact the placement of quality concrete. Cold weather conditions are generally taken to mean when the air temperature has fallen to, or is expected to fall below, 40 °F (4.4 °C) during the protection period that can cause damage to the young concrete if it freezes while it is still fresh or before it has developed adequate strength (generally taken to be a minimum compressive strength of 500 lbf/in^2 [3.4 MPa]) (ACI 2016).

Both hot and cold weather conditions present significant paving challenges, and additional guidance and information on hot and cold weather concrete is available from the American Concrete Institute (ACI 2010, ACI 2016). Also, as indicated previously, the HIPERPAV software program may be useful in evaluating the impacts of various environmental conditions on the potential for early-age cracking; the user's manual for the current version of the software is available from the FHWA (Ruiz et al. 2015).

Hot weather paving recommendations to minimize cracking are as follows:

1. Do not exceed the maximum allowable w/cm ratio or the manufacturer's maximum recommended dosage of any admixture.
2. Consider the use of retarding admixtures if their performance has been verified during trial batches.
3. Substitute slag cement, Class F fly ash, or natural pozzolans for part of the portland cement, as these materials hydrate more slowly and generate a lower heat of hydration. Note that certain Class C fly ashes with high calcium and aluminum contents may cause problems associated with premature stiffening.
4. Correct low air contents by increasing the dosage of air entraining admixture.
5. Prevent early-age thermal cracking by ensuring that the temperature of the plastic concrete is as low as practical. Cool the aggregates by sprinkling with water or chilling the mixing water.
6. Consider painting the mixing and transporting equipment white or a light color to minimize the heat absorbed from the sun.
7. In extreme conditions, schedule concrete placements for nighttime.
8. Moisten the base before the concrete is placed to keep the temperature down and to keep it from absorbing water from the concrete.
9. Place and finish the concrete as rapidly as possible and apply the curing compound as early as possible. Note also that the use of a white curing compound will reflect the sun's heat.
10. Steps should be taken during hot weather to reduce the rate of evaporation from the concrete (see the 2010 ACI publication on hot weather concreting for additional information on estimating evaporation rates). When high evaporation rates are anticipated, provide fog spraying or use an approved evaporation retardant as appropriate.

(Adapted from Kohn et al. 2003)

Cold weather paving recommendations to minimize cracking are as follows:

1. The required dosage of air entraining admixture will likely be lower than the dose at normal temperatures.
2. Use an accelerating admixture that conforms to ASTM C494 Type C or E nonchloride accelerator if its performance has been verified during trial batches.
3. Do not use admixtures containing added chlorides. Also, do not use calcium chloride.

4. Free aggregates of ice, snow, and frozen lumps before placing in the mixer.
5. The temperature of the mixed concrete should not be less than 50 °F (10 °C)—65 °F (18 °C) if an SCM is used—which may be achievable by heating the mixing water.
6. Generally speaking, concrete should not be placed when the temperatures of the air at the site or the surfaces on which the concrete is to be placed are less than 40 °F (4.4 °C), unless steps are taken to achieve a minimum mix temperature of 50 °F (10 °C).
7. Reduce heat loss with insulating blankets. These may be temporarily rolled aside to allow for joint sawing.
8. The concrete temperature should be maintained at 50 °F (10 °C) or above for at least 72 hours after placement and at a temperature above freezing for the remainder of the curing time (when the concrete attains a compressive strength of 3,000 lbf/in^2 [20.7 MPa]).

(Adapted from Kohn et al. 2003)

Transverse and Diagonal Cracking Caused by Settlement and Poor Support

Transverse and diagonal cracking can also be caused by settlement of the underlying foundation and the resulting poor support (see Figure 5.4). The foundation collectively refers to the base (layer immediately beneath the slab), subbase (layer or layers beneath the base and above the subgrade), and the prepared subgrade, and each may adversely affect the level of support provided to the concrete slab.

Figure 5.4 Settlement cracking

Dan DeGraaf, Michigan Concrete Association

Cause

Settlement and poor support conditions can result from a number of factors, including the following.

Poor compaction and nonuniform support: Settlement beneath a slab may be the result of poor or inadequate compaction of the underlying foundation materials. This settlement leads to increases in slab deflections and slab stresses, and can result in transverse and diagonal cracking. It is common to observe settlement cracking over culverts or in utility patches because of the difficulty in achieving high levels of compaction in those areas. In addition, areas exhibiting nonuniform support (for example, due to variability in subgrade materials, segregation of granular base course materials, or variability in compaction) can lead to the development of transverse or diagonal cracking in the concrete pavement.

Base erodibility and loss of support: Some base types are highly susceptible to pumping and erosion, a process in which the saturated base material is ejected from beneath the slab under traffic loading. Consequently, that portion of the slab is no longer fully supported, producing high bending stresses under heavy loading that can cause the slab to crack in the vicinity of the eroded area. Generally, most stabilized base courses are more erosion-resistant than nonstabilized bases.

Subgrade volume stability: Some subgrades are susceptible to changes in their volumes that affect the foundational support provided to the concrete pavement system. These changes commonly may be the result of swelling soils or frost heave. Swelling soils are ones that swell and shrink due to changes in the moisture content, and are more commonly associated with heavy clay materials (e.g., AASHTO A-6 and A-7 classifications). Frost heave refers to the distortion or expansion of the subgrade as a result of the growth of ice lenses under freezing temperatures, and requires the contribution of three factors:

- A sufficiently cold climate to allow freezing temperatures to penetrate into the subgrade
- A supply of moisture into the freezing zone
- A susceptible soil (typically low plasticity silts, which exhibit a sufficient combination of capillarity and permeability to feed the growth of the ice lenses) (ACPA 2007a)

Both swelling soils and frost-heave actions lead to nonuniform support conditions that can contribute to the development of slab cracking. More details on frost heave and swelling soils can be found in Chapter 12.

Prevention

A summary of general recommendations on addressing settlement and support issues is provided in Table 5.4.

Table 5.4 Considerations for addressing transverse and diagonal cracking caused by settlement or poor support

Foundation Element	Aspect	Issue	Considerations
Base and Subgrade	Compaction or Density	Poor or inadequate compaction can lead to settlement of the base, reducing the support provided to the slab	Ensure that adequate compaction to the specified target density is achieved for all unbound foundation layers (base, subbase, subgrade)
Base and Subgrade	Uniformity of Support	Areas of nonuniformity in the base, subbase, or subgrade can lead to cracking	Ensure base course is homogenously blended when placed Undercut, replace, or stabilize soft spots in subgrade Consider subgrade stabilization for plastic soils
Base	Base Erodibility	Pumping and erosion of base beneath slab leads to unsupported conditions	Use dowel bars at transverse joints Use less erodible base materials, particularly for heavy truck traffic facilities If aggregate base courses are used, consider limiting the amount of fines to 10% or less (with consideration given to stability requirements for construction trafficking) Provide drainage by installing edge drains or by daylighting the aggregate base
Subgrade*	Swelling Soils*	Subgrade volumetric changes due to variations in subgrade moisture contents	For small areas, remove and replace Compact at 1 to 3% above optimum moisture content (AASHTO T 99) Consider use of stabilization and membranes
Subgrade*	Frost Heave*	Subgrade volumetric changes due to frost penetration and growing ice lenses in subgrade	Compact slightly wet of optimum moisture content (AASHTO T 99) Use non-frost-susceptible materials within depth of frost penetration Cover frost-susceptible soil with sufficient thickness of non-frost-susceptible material

* More detailed information provided in Chapter 12

Regarding the selection of an appropriate base type for a paving project, note that there are contradictions in the ability of different base types to meet some of these requirements. For example, stiffer bases contribute to higher curling stresses and frictional forces yet are more resistant to erosion and loss of support. Nevertheless, a wide variety of base types have been used successfully beneath concrete pavements provided that uniformity of support is provided and other features and characteristics of the base are considered properly.

Transverse Cracking Caused By Excessive Slab Lengths

The layout and spacing of transverse joints in concrete pavements is one of the most common factors that can influence the development of transverse and diagonal cracking on concrete pavements (see Figure 5.5).

Figure 5.5 Transverse cracking caused by excessive slab lengths (20-foot panel)

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Generally speaking, thicker slabs can accommodate longer joint spacings, and an oft-cited rule of thumb for the maximum joint spacing is that the slab length (expressed in feet) should be no more than twice the slab thickness (expressed in inches). For example, an 8-inch slab should have a joint spacing of no more than 16 feet. However, this is an overly simplistic guide, and does not account for a number of other critical factors, such as the stiffness of the base and the frictional characteristics between the base and slab.

The AASHTOWare Pavement ME design procedure directly considers joint spacing as an input into the design process and considers a range of other design and environmental factors to enable design engineers to determine suitable joint spacings for a set of specific design conditions (AASHTO 2015). Today, most agencies have adopted a maximum transverse joint spacing of 15 feet for their JPCP highway pavements.

Cause

As previously described, tensile stresses in concrete pavements develop shortly after placement as the slab responds to temperature changes and moisture losses. A system of joints is created in concrete pavements to alleviate those stresses before they exceed the strength of the young concrete.

For most highway applications, joints are provided by creating a plane of weakness at the planned joint location by saw cutting into the concrete but they may also be created through forming, notching, or other methods. An effective spacing and layout of joints not only compels the concrete to crack at that planned location but also provides a uniform, more easily maintained and aesthetically pleasing slab partition. Thicker slabs are more resistant to cracking caused by traffic loading effects but require deeper saw cuts or forming to ensure the development of the joints at the planned locations.

The magnitude of thermal curling, moisture warping, and shrinkage stresses all increase with increasing joint spacings. If the transverse joints are spaced too far apart, those stresses can exceed the strength of the concrete and result in transverse and diagonal cracking. Two characteristics of the underlying base course in particular can play a significant role in the development of those critical slab stresses:

- **Base stiffness**: The stiffness of the base influences the behavior of the slab under traffic and environmental loading. Stiff base courses, such as lean concrete or cement-treated bases, provide strong structural support but resist slab deformations generated by thermal and moisture gradients in the slab, thus leading to increased stress development. More information on curling and warping is provided in Chapter 10.
- **Slab-base friction**: High friction levels (or perhaps even bonding) between the concrete slab and the underlying base course can contribute to cracking development. This is because the friction restrains the volume change that the concrete undergoes shortly after placement. The higher friction levels also serve to shorten the time available for the saw cutting of transverse joints. Aggregate base courses exhibit lower frictional levels than stabilized base courses. In the case of permeable treated bases, it has also been observed that the concrete can actually penetrate into the permeable base and restrain slab movement, leading to the development of uncontrolled cracking.

Prevention

The jointing pattern for a pavement structure must be effectively established to minimize the potential for uncontrolled cracking. The objective in selecting transverse joint spacings for JPCP designs is that they be short enough to inhibit the development of mid-panel transverse and transverse cracks, yet long enough to reduce joint construction and maintenance costs. However, joint design and layout are interlinked with slab thickness, base stiffness, and slab-base frictional conditions, as well as concrete properties and prevailing climatic conditions.

Although joint spacings traditionally were selected based on agency standards or on local experience, AASHTOWare Pavement ME Design software includes it as a direct input so that the interactions between these variables are considered (AASHTO 2015). As mentioned earlier, most highway concrete pavements (in the thickness range of 10 to 12 inches employ a maximum transverse joint spacing of 15 ft (4.6 m), with thinner pavements or pavements constructed on very stiff bases courses requiring shorter transverse joint spacings. Table 5.5 presents some considerations related to the use of stabilized base courses as related to slab cracking.

Table 5.5 Base considerations to prevent transverse and diagonal cracking

Foundation Element	Aspect	Issue	Considerations
Base	Base Stiffness	Stiffer base courses can lead to higher curling stresses	Limit 7-day strength of lean concrete bases to 1200 lbf/in^2 or less Use debonding material or geotextile separator layer between slab and stabilized (lean concrete or cement-treated) base Reduce joint spacing (15 ft or less for most highway pavements)
Base	Slab–Base Friction	High friction levels (or even bonding) between the slab and base can restrain the slab and lead to cracking	Use debonding material between slab and stabilized bases (lean concrete or cement-treated) Reduce joint spacing (15 ft or less for most highway pavements)

Transverse and Diagonal Cracking at Intersections

Urban roadway environments present significant jointing challenges because of the presence of intersecting roadways, emerging and receding traffic/turn lanes, median islands and curb/gutter installations, and in-pavement utilities. If jointing is not carefully planned and laid out, random cracking can develop that will significantly detract from the performance of the pavement. Figure 5.6 depicts cracking observed in urban concrete intersections and highlights the need for special considerations when paving in this type of environment.

Figure 5.6 Cracking at urban intersections

a. Cracking due to poor joint layout
John Donahue, Missouri DOT

b. Cracking caused by lack of curb saw cut
Dale Harrington, HCE Services

c. Cracking caused by lack of jointing to isolate a manhole

Cause

The primary cause for transverse and diagonal cracking in urban concrete intersections is related to the layout of the joints. While most agencies will work to maintain the same maximum joint spacing used in their conventional concrete paving, if that spacing is rigidly maintained, there could be issues in matching up with existing pavement structures, curbs, and adjacent traffic lanes. Furthermore, the layout of joints at sharp, acute angles to one another or not working to maintain largely square slabs can create stress concentrations that can lead to the development of slab cracking.

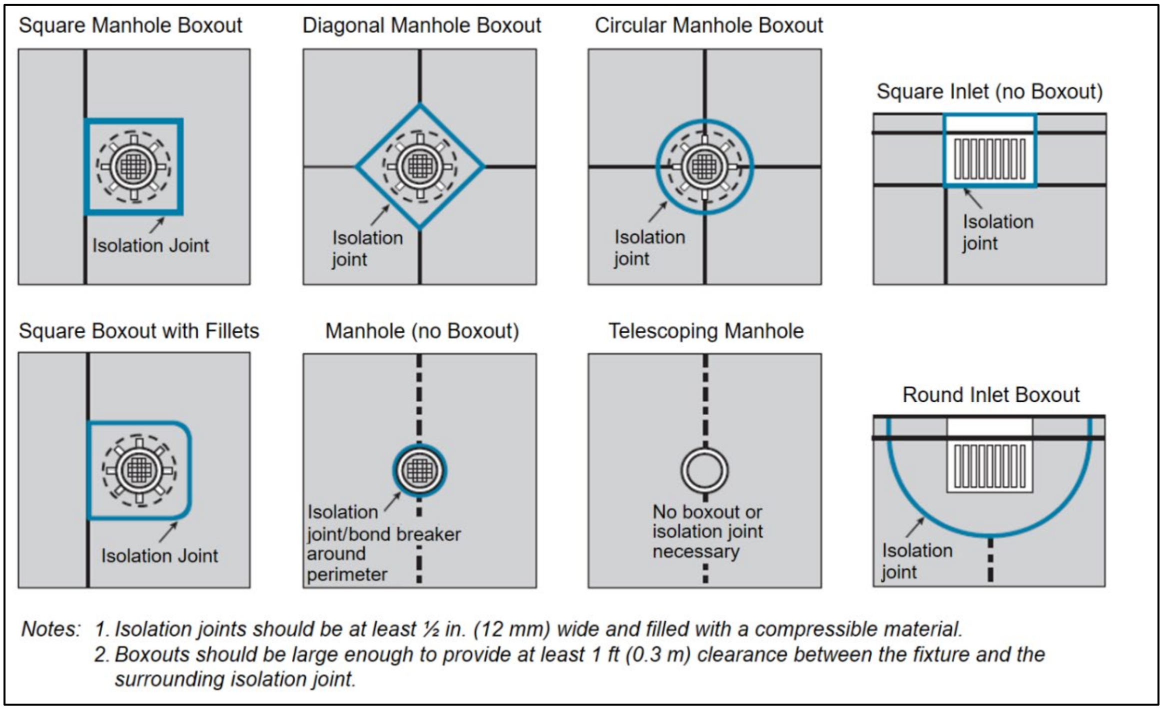
Prevention

General recommendations for addressing unique jointing situations in urban environments include the following (Uhlmeyer 2003, ACPA 2007b):

- Prepare a jointing plan.
- Match and align joints.
- Avoid joint intersection angles less than 60º.
- Keep slabs short and relatively square.
- Avoid odd-shaped slabs.
- Accommodate in-pavement structures through jointing and boxouts.
- Make field adjustments as needed.

A maximum joint spacing for the slab thickness and base conditions should be used in the general layout of the transverse joints, and typically longitudinal joints will be laid out to match traffic lane widths. However, some adjustments to those dimensions may be needed to help match existing pavements and to accommodate in-pavement structures such as manholes, utility valves, and drainage inlets. These in-place structures will often require boxouts with a perimeter isolation joint to accommodate movements, and the boxouts themselves will need to be carefully jointed to avoid sympathy cracks from boxout corners. In lieu of boxouts, some contractors elect to wrap the in-place structure with a pliable expansion joint filler or may cast the structure directly into the concrete (ACPA 2007b). Figure 5.7 provides some details for boxing out in-pavement structures.

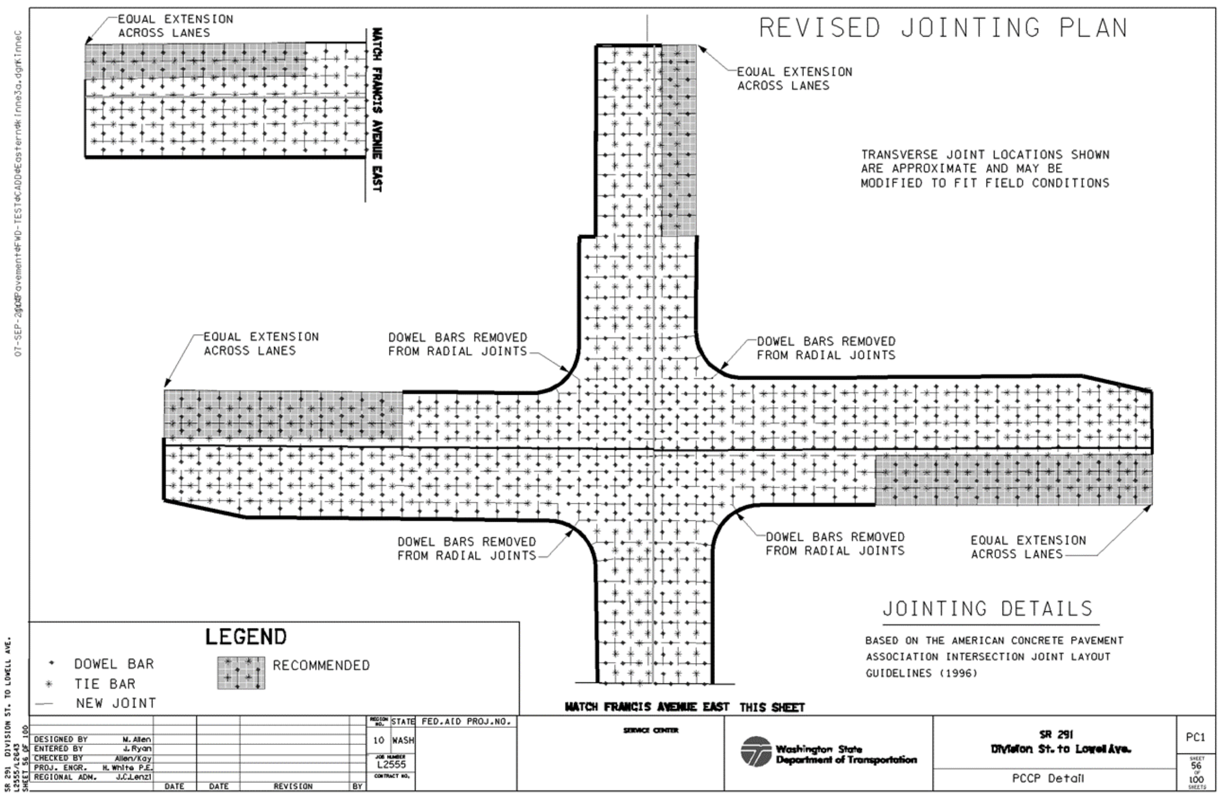
Figure 5.7 Details for boxing out in-pavement structures



ACPA 2007b

The development of a detailed jointing plan is a key element in the successful construction of concrete intersections, or any other facility where jointing patterns may become complex. Even with a jointing plan, some field adjustments may still be required to minimize the occurrence of uncontrolled cracking. Figure 5.8 shows an example jointing plan.

Figure 5.8 Example jointing plan from a contract document



Uhlmeyer 2003, Washington State DOT

Transverse Cracking Due to Poor Sawing Practices

For most highway paving, joint sawing is the common method of creating the transverse and longitudinal joints in the concrete pavement. The saw cut is not through the entire thickness of the slab but rather through a portion of it, thereby creating a plane of weakness that encourages the crack to occur at that location. The depth of the saw cut in a concrete slab has commonly been one-quarter of the slab thickness for transverse joints and one-third of the slab thickness for longitudinal joints but some agencies have moved toward cutting all joints to a depth of one-third of the slab thickness.

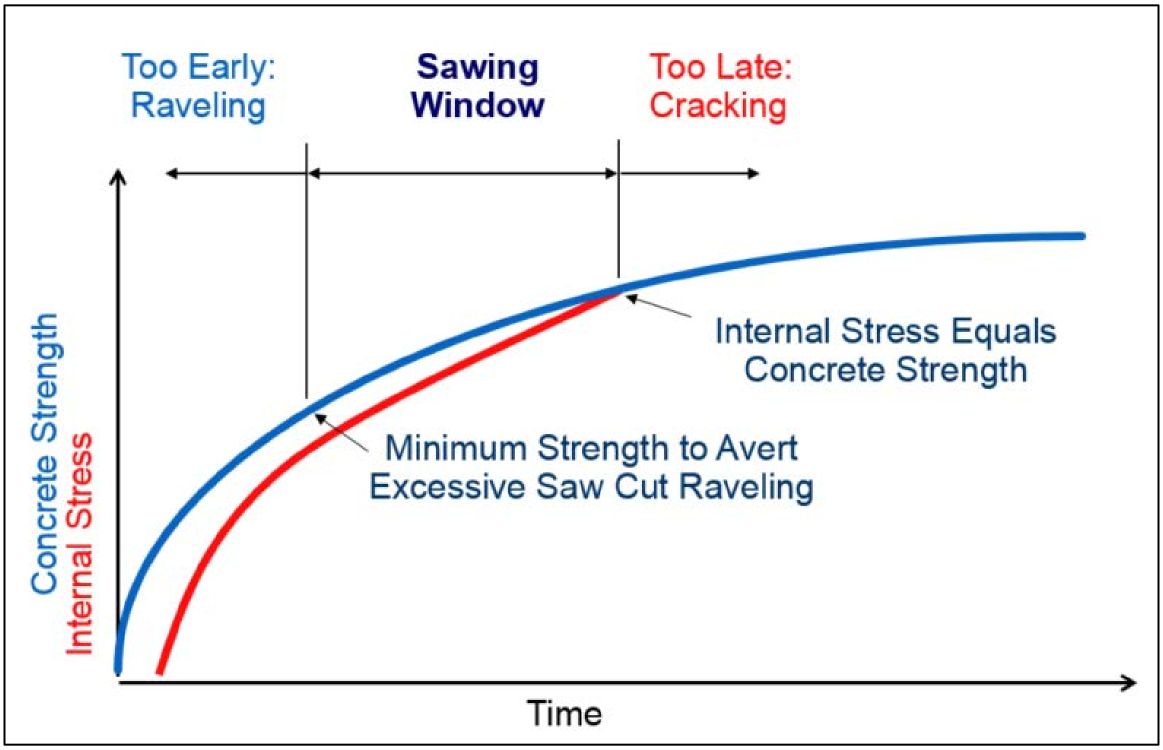
Cause

The establishment of weakened plane joints in a concrete pavement is a critical aspect of concrete pavement construction both in terms of depth and timing. Joints that are created or formed too late (after excessive stresses have already developed in the slab) or that are not created deep enough run the risk of developing random, uncontrolled cracking.

While the depth of the joint creation is a straightforward parameter, determining the timing of the saw cutting operation is somewhat challenging. In general, the joint sawing should begin as soon as possible after the slab has achieved adequate strength, and under most normal conditions this may be between 4 and 12 hours after placement (ACPA 1991). The goal in timing the saw cutting operation is to not saw too soon, which will cause raveling of the concrete, nor too late, which may result in random cracking. The acceptable period of time is often referred to as the "saw cutting window," which is illustrated conceptually in Figure 5.9.

In practice, the appropriate saw cutting time can vary considerably depending on environmental conditions, mix design parameters, curing, and other factors, and falls under the responsibility of the contractor.

Figure 5.9 Joint saw cutting window



American Concrete Pavement Association

Figure 5.10a shows a proper saw cut and the crack that has developed beneath it, whereas Figure 5.10b illustrates the development of a parallel crack that may be the result of late sawing or inadequate depth of sawing.

Figure 5.10 Saw cuts for weakened plane joint

a. Proper weakened plane saw cut and induced crack
Jeff Uhlmeyer, Washington State DOT

b. Slab cracking occurring parallel to weakened plane saw cut
Derek Tompkins, American Engineering Testing, Inc.

Prevention

The prevention of pavement cracking due to saw cutting issues is addressed by sawing or forming the joints to the proper depth and by creating those joints within the saw cutting window. The duration of the saw cutting window is highly variable and depends on a number of factors, including the following (ACPA 2002).

- Weather (environmental)
    - Sudden temperature drop or rain shower
    - Sudden temperature rise
    - High winds and low humidity
    - Cool temperatures and cloudy
    - Hot temperatures and sunny
- Pavement and base
    - High friction between the base and slab
    - Bond between the base and slab
    - Dry surface
    - Porous aggregate subbase materials
    - Presence of existing pavement adjacent to new pavement
- Concrete mixture
    - High water demand
    - Rapid early strength
    - Retarded set
    - Cementitious content and composition
    - Supplementary cementitious materials
    - Fine aggregate (fineness and grading)
    - Coarse aggregate (maximum size and percentage)

Experienced saw cutting crews assess the suitability of the concrete for sawing through a simple "scratch test" in which the surface is abraded with a nail or knife blade. As the surface hardens the scratch depth decreases, and in general if the scratch removes the surface texture then it is likely too early to saw (Taylor et al. 2006). The use of the HIPERPAV software program or ongoing monitoring of the slab strength can also be used to help identify appropriate sawing times. The contractor must also select sawing equipment that is suitable to the thickness of the slab and a saw blade that is compatible with the concrete mixture and aggregate type. Sawing of both the transverse and longitudinal joints should commence at the same time.

One critical item for saw cutting crews is the recognition that the concrete can set up at different rates through the thickness of the slab. This can be a particular issue in late season paving, in which the bottom of the slab sets up before the top of the slab because of the warmer ground temperatures. If that is the case, the saw cutting may need to begin earlier than what would otherwise be suggested by the scratch test.

As with all concrete construction, proper curing is essential to retaining moisture in the slab and ensuring effective strength development. This is commonly performed by applying liquid membrane-forming curing compounds on the pavement surface at rates between 100 and 200 ft^2/gal (2.45 and 4.90 m^2/L), depending on the type of pavement. For example, a rate of 200 ft^2/gal (2.45 m^2/L) is used for most normal paving operations whereas a rate of 100 ft^2/gal (4.90 m^2/L) is often used for thin concrete overlays (Taylor et al. 2006). The curing compound should be placed as soon as possible after surface finishing to both the surface and sides of the slab. In some instances, the use of evaporation retarders, which are sprayable solutions that are applied to fresh concrete to reduce the rate of surface moisture evaporation prior to finishing, may be appropriate (but these should not be used as finishing aids).

Transverse Cracking Caused by Traffic Loading

Traffic loading can play a role in the development of transverse and diagonal cracking in a concrete pavement and can influence both early-age and long-term cracking behavior.

Cause

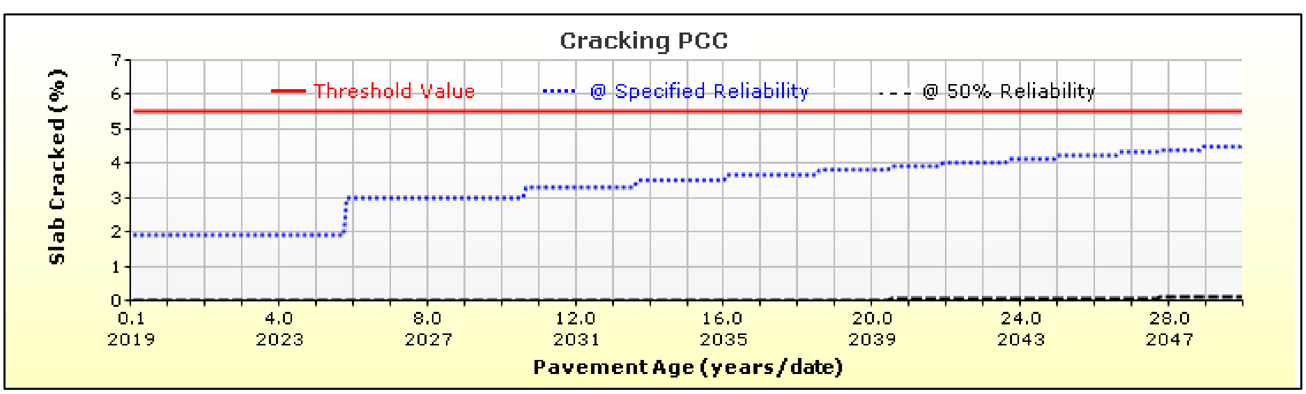
Traffic loading can cause transverse and diagonal cracking on a concrete pavement either through the application of a single load repetition (in which the stress induced by the traffic load exceeds the strength of the concrete) or, much more commonly, through the repeated applications of traffic loading (in which lower stress levels are induced but repeated many times over an extended time period). The latter case, referred to as fatigue cracking, serves as the basis for most mechanistic-based design procedures in that a suitable slab thickness is determined to minimize the magnitude of the stresses so that cracking is maintained to an acceptable level over the design period and for the prevailing design conditions (climate, materials, foundation support, etc.).

Traditional fatigue cracking analysis has assumed that the transverse and diagonal cracking initiates from the bottom of the slab but more recently it has been recognized that transverse and diagonal slab cracking can also initiate from the top of the slab depending on environmental impacts, curing conditions, and loading configurations. Environmental conditions can also affect long-term concrete behavior and influence the development of pavement cracking. For example, thermal curling and moisture warping can produce stresses in the slab that may be additive to load-induced stresses, thereby contributing to the development of fatigue cracking.

Prevention

Fatigue-based transverse and diagonal cracking can be best addressed through the development of competent pavement structural designs that have been developed for the prevailing environmental conditions, foundation support characteristics, mix design parameters, and expected traffic loading repetitions over the design period. The AASHTOWare Pavement ME Design software, as documented by AASHTO (2015), can be used to develop pavement designs that will limit traffic-induced transverse and diagonal cracking to user-defined acceptable levels, and can directly accommodate critical factors such as foundation support and friction, curling, and joint spacing. Figure 5.11 gives an example of the concrete pavement cracking (transverse and diagonal) output from the AASHTOWare Pavement ME Design software.

Figure 5.11 Example slab cracking output from AASHTOWare Pavement ME Design software



Transverse and Diagonal Sympathy Cracks

It is common for joints in adjacent lanes, shoulders, or roadways to create “sympathy” cracks in a concrete pavement (see Figure 5.12).

Figure 5.12 Transverse sympathy cracks

a. Sympathy crack from transverse joint in adjoining lane
Jeff Uhlmeyer, Washington State DOT

b. Sympathy crack from joint aligning with catch basin
Jeff Uhlmeyer, Washington State DOT

These cracks are usually observed in urban streets and intersections but may also occur in mainline highway paving if the lanes (or shoulder) are paved separately or if the joint spacings are not aligned across separately placed paving pours.

Cause

Sympathy cracks can occur in a concrete pavement where a joint (or crack) in an adjacent slab terminates up against an unjointed/uncracked slab panel. That joint (or crack) in the adjacent slab opens and closes in response to temperature changes, which creates a tensile stress concentration at the face of the uncracked slab that can result in cracking if the strength of the concrete is exceeded. The timing of the occurrence of these cracks varies depending on the environmental conditions, base type and friction, and joint type.

Prevention

As with jointing at intersections, careful consideration should be given to the layout of joints across multiple pavement lanes (including any concrete shoulder that may be constructed). Joints should be aligned across all lanes and joints in the pavement slabs should be carried through adjacent shoulders or curbs to prevent sympathy cracks. Some field modifications may be needed to ensure that the various jointing requirements are met in order to reduce the risk of uncontrolled cracking. Where it is not possible to match joints across lanes, an isolation joint may be needed to prevent sympathy cracking.

Table 5.6 summarizes the key points related to the causes and prevention of transverse and diagonal cracking in concrete pavements.

Table 5.6 Summary of causes and prevention or mitigation of transverse and diagonal cracking

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Transverse Cracking Due to Volumetric Changes	Volume changes in the slab caused by concrete mixture properties and environmental factors	N/A	Avoid high-shrinkage mixtures (high cement contents, high CTE) Use well-graded aggregates Consider use of water-reducing admixtures	Employ appropriate hot- and cold-weather paving practices when paving under extreme weather conditions Consider late afternoon or nighttime paving during hot conditions Consider thermal blankets during cold conditions Avoid large temperature changes as concrete is placed and cures, or work to minimize effects Employ HIPERPAV or other software to determine cracking risk for potential paving scenarios

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Transverse and Diagonal Cracking Caused by Settlement and Poor Support	Increased deflections and slab stresses as the result of poor compaction, nonuniform support, erosion, and subgrade volumetric changes	Specify adequate compaction levels for subgrade and all unbound bases/subbases Consider effects of erodibility when specifying base course Devise plan for addressing expansive and frost-susceptible soils (removal, compact wet of optimum, stabilization, and cover layer) Consider effects of swelling soils and frost heave on crack potential	Consider use of erosion-resistant materials	Ensure field control of foundation layers to provide adequate compaction and uniformity of support, including in culvert areas and utility trenches
Transverse Cracking Caused By Excessive Slab Lengths	Longer slab lengths increase critical curling, warping, and shrinkage stresses in the slab that can lead to cracking	Employ suitable joint spacing for climate conditions, foundation support and friction, and slab thickness	Avoid high-shrinkage mixtures (high cement contents, high CTE) Use well-graded aggregates Consider use of water-reducing admixtures	N/A

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Transverse and Diagonal Cracking at Intersections	Poor layout of joints at intersections: Use of acute joint angles Joints terminating against adjacent slabs Odd-shaped slabs Utilities not jointed properly	Develop appropriate jointing plan for project scenario (including the accommodation of in-pavement structures and utilities)	N/A	Follow jointing plan, making field modifications as appropriate Saw cut joints within the window of opportunity Saw cut joints to the specified depth Monitor early-age strength development Employ HIPERPAV or other software to determine cracking risk for potential paving scenarios
Transverse Cracking Due to Poor Sawing Practices	Inadequate saw cut depth or late sawing	Employ suitable joint spacing for climate conditions, foundation support and friction, and slab thickness	N/A	Saw cut joints within the window of opportunity Saw cut joints to the specified depth Monitor early-age strength development Employ HIPERPAV or other software to determine cracking risk for potential paving scenarios

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Transverse Cracking Caused by Traffic Loading	Excessive stresses in the slab due to truck loading and slab curling/warping effects	Determine appropriate slab thickness for traffic levels Employ suitable joint spacing for climate conditions, foundation support and friction, and slab thickness	Avoid high-shrinkage mixtures (high cement contents, high CTE) Use well-graded aggregates Consider use of water-reducing admixtures	Construct pavement to proper thickness tolerances Keep construction traffic and other early loading away from slab edges Monitor early-age strength development
Transverse and Diagonal Sympathy Cracks	Stress concentration in slabs caused by movement of terminating joints in adjacent slabs that open and close due to temperature changes	Prepare appropriate jointing plan for paving slabs and adjacent concrete shoulders and curb/gutter Accommodate in-pavement structures and utilities Isolate new slab from adjacent with isolation joint	N/A	Follow jointing plan, making field modifications as appropriate Saw cut joints within the window of opportunity Saw cut joints to the specified depth Monitor early-age strength development Employ HIPERPAV or other software to determine cracking risk for potential paving scenarios

6. Treatment and Repairs

General repair methods and maintenance approaches to address transverse and diagonal cracking distress in concrete pavements are described in the following sections. It should be noted that in some cases, the cracking distress does not pose a significant performance issue and may be best left untreated (e.g., if the cracking is tight or if the pavement is exposed to low truck traffic levels).

Repairs

Full-Depth Repair (FDR) and Slab Replacement

Full-depth repair (FDR) and slab replacement are commonly used to address deteriorated transverse and diagonal cracks in concrete pavements. These repairs consist of the removal of isolated deteriorated areas through the entire thickness of the existing slab and the replacement with a concrete replacement material. The *Concrete Pavement Preservation Guide* (Smith et al. 2014) provides details on the design and construction of effective FDRs and slab replacements.

Dowel Bar Retrofit

Dowel bar retrofit of transverse or diagonal cracks is an alternative to FDR or slab replacement. This process installs dowel bars in slots across the crack, helping to provide good load transfer across the crack and reducing deflections, pumping, and further crack deterioration. Diamond grinding is often performed in conjunction with dowel bar retrofit to restore the overall ride quality. More details on the use of retrofitted dowel bars is available in the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

Diamond Grinding

Diamond grinding involves the removal of a thin layer of hardened concrete pavement surface using a self-propelled machine outfitted with a series of closely spaced diamond saw blades mounted on a rotating shaft. Diamond grinding does not directly address slab cracking but may be used in conjunction with the treatments listed above to eliminate faulting and restore overall pavement rideability.

Maintenance

Crack Sealing

The sealing of transverse and diagonal cracks can reduce the amount of water that infiltrates the pavement system and consequently may help prevent or minimize the further breakdown and deterioration of the crack; the operation may also help to reduce the potential for spalling. Crack sealing may be performed on cracks up to about 1/2-inch wide but it is generally recommended that tight, low-severity cracks be left unsealed. Overall, this may be a short-term solution to minimizing water infiltration and does nothing directly to address the deterioration that has already occurred.

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CHAPTER 6. LONGITUDINAL CRACKING

1. Description

Cracking refers to a distinct full-depth fracture in a jointed concrete pavement. Longitudinal cracking, the focus of this chapter, is oriented roughly parallel to the pavement centerline or lane-shoulder joint. Other forms of slab cracking may also develop transversely (cracks that are roughly perpendicular to the centerline), diagonally (cracks at angles of about 30 to 60 degrees with respect to the centerline or to transverse joints), or at panel corners (diagonal cracks that extend from a transverse joint to a longitudinal joint, with legs no longer than about 6 feet along either joint).

Regardless of orientation, these types of cracks are differentiated from map cracking and other surficial or durability-related cracking in that they are distinct cracks that typically extend through the entire thickness of the slab. Moreover, these cracks can develop in conjunction with one another to produce what is often referred to as a shattered or broken slab (in which the slab is divided into three or more pieces).

Chapter 5 provides detailed information about transverse and diagonal cracking and Chapter 7 provides detailed information about corner cracking. This chapter focuses primarily on longitudinal cracking.

Figure 6.1 presents some typical photos of longitudinal cracking.

Figure 6.1 Longitudinal cracking in concrete pavements

a. Longitudinal crack (with 0.2 inch faulting) on JPCP
John Donahue, Missouri DOT

b. Longitudinal crack in outer wheel path of the travel lane
Long Term Pavement Performance Program (LTPP)

c. Longitudinal cracking due to late sawing
Dan DeGraaf, Michigan Concrete Association

d. Longitudinal crack over dowels at JPCP transverse joint
Mark B. Snyder, used with permission

e. Longitudinal crack along centerline joint with possible late sawing
Mark B. Snyder, used with permission

2. Severity

Longitudinal cracking is classified as a distress for jointed plain concrete pavement (JPCP), according to the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014). Table 6.1 summarizes the long-term pavement performance (LTPP) severity levels and measurement methods for longitudinal cracking in these cases.

Table 6.1 Severity levels of longitudinal cracking

Distress	Description and Severity Levels	Measurement
Longitudinal Cracking – Jointed Concrete Pavement (JCP)	Cracks that are predominantly parallel to the pavement centerline **Low**: Crack widths less than 0.125 in. (3 mm), no spalling, and no measurable faulting or well-sealed and with a width that cannot be determined **Medium**: Crack widths greater than 0.125 in. (3 mm) but less than 0.50 in. (13 mm); or with spalling less than 3 in. (75 mm); or faulting up to 0.50 in. (13 mm) **High**: Crack widths greater than 0.50 in. (13 mm) or with spalling greater than 3 in. (75 mm) or faulting greater than 0.50 in. (13 mm)	Record the length of longitudinal cracking at each severity level. Also record the length of longitudinal cracking with sealant in good condition at each severity level. Sealant is not considered to be in good condition unless at least 3 ft (1 m) of continuous sealant in good condition is present. In cases where a crack is less than 3 ft (1 m) long, the sealant must be present and in good condition over the entire length of the crack.
Longitudinal Cracking – Continuously Reinforced Concrete Pavement (CRCP)	Cracks that are predominantly parallel to the pavement centerline **Low**: Crack widths less than 0.125 in. (3 mm), no spalling, and no measurable faulting or well-sealed and with a width that cannot be determined **Medium**: Crack widths greater than 0.125 in. (3 mm) but less than 0.50 in. (13 mm) or with spalling less than 3 in. (75 mm) or faulting up to 0.50 in. (13 mm) **High**: Crack widths greater than 0.50 in. (13 mm) or with spalling greater than 3 in. (75 mm) or faulting greater than 0.50 in. (13 mm)	Record the length of longitudinal cracking at each severity level. Also record the length of longitudinal cracking with sealant in good condition at each severity level. Sealant is not considered to be in good condition unless at least 3 ft (1 m) of continuous sealant in good condition is present. In cases where a crack is less than 3 ft (1 m) long, the sealant must be present and in good condition over the entire length of the crack.

Source: Modified from Miller and Bellinger 2014

3. Testing

To accurately determine the depth of crack penetration, and to evaluate the underlying support conditions, field testing including coring and the falling weight deflectometer (FWD) must be administered. If there are concerns regarding temperature or shrinkage characteristics, laboratory testing may be performed. More detailed information regarding testing can be found in Chapter 19.

Field Tests

When longitudinal cracking is present, field testing may help to determine possible causes of the distress and to identify appropriate preservation or rehabilitation solutions. These additional field tests are described below.

Coring and Core Examination

Concrete core samples can be retrieved at the cracks to determine the depth of the crack penetration through the slab, as well as to verify the thickness of the slab. This also provides the opportunity to observe the tortuosity of the cracking through the concrete and whether the crack proceeds around or through the coarse aggregate particles. Cracking around an aggregate particle suggests an early cracking mechanism while cracking through the aggregate (when paste bond strength has increased) suggests later age cracking (Walker et al. 2006), although these guidelines are not absolute. Coring can also be used to assess the condition and potential bond/friction characteristics of the underlying base below the crack.

Cores of adjacent sawed longitudinal joints should also be retrieved to measure the depth of the joint sawing (for comparison with specification requirements) and to determine whether cracks have activated in the weakened plane below the saw cut. These cores may be more useful for projects exhibiting early-age cracking, with coring of transverse joints appropriate when early-age transverse and diagonal cracking have occurred. If present, any steel reinforcement in the slab may be inspected for signs of corrosion or deterioration.

Straightedge or String Line Testing

A straightedge can provide an indication of longitudinal crack depth when the cracking is caused (at least in part) by a void or other loss of support. The straightedge should be rigid and sufficiently long enough to be able to establish the general slope of the intact portion of the slab and to project far enough across the cracked portion of the slab to reveal a break in the slopes of the two slab fragments at the crack. A break in slope may also be visible by anchoring a string line or cable to the surface of the uncracked portion of the slab several feet from the crack and pulling the string line taut along the surface and over the cracked portion of the slab. If there is no apparent break in the slope, the longitudinal crack may not yet extend through the full slab thickness where a void is present or is not due to a loss of support. Other types of testing (described below) can provide an indication of whether a void or other loss of support is present.

FWD Testing

Deflection testing using the FWD can be performed to evaluate the underlying support conditions to determine if they may have contributed to the development of the cracking. In addition, the FWD can be used to assess the load transfer characteristics of the cracks and to detect the presence of voids beneath the slab.

Ground-Penetrating Radar (GPR) and Ultrasonic Echo Imaging

GPR and ultrasonic testing devices may be useful in detecting the presence of voids in areas where longitudinal top-down cracking has initiated but the panel has not yet fully broken and settled.

Laboratory Tests

Laboratory testing may be conducted if there are concerns about the temperature and shrinkage characteristics of the slab potentially contributing to the development of slab cracking. This could include evaluation of the coefficient of thermal expansion (CTE), as well as petrographic analyses to provide insight concerning mix proportioning parameters (e.g., water to cementitious material [w/cm] ratio, cement content, supplementary cementitious materials [SCM] content, air void system characteristics, etc.).

4. Identification of Causes

Concrete pavements generally crack when tensile stresses in the slab exceed the tensile strength of the concrete. These conditions develop most frequently at critical locations (i.e., the top or bottom of the panel at edges and/or corners) where slab stresses tend to be higher because of load placement and/or support conditions, resulting in crack initiation at the critical location(s) and propagation through and along the slab.

To minimize the potential for uncontrolled slab cracking, a system of joints—both transverse and longitudinal—is introduced into jointed concrete pavements to promote the development of the cracks at predetermined locations to control crack locations, facilitate their maintenance and to provide mechanical devices for enhanced load transfer (e.g., smooth dowels and deformed tie bars).

Many factors can contribute to the inducement of slab stresses that result in longitudinal cracking. Table 6.2 provides a summary of these factors.

Table 6.2 Summary of physical and material/chemical causes of longitudinal cracking

Distress	Category	Causes
Longitudinal Cracking	Physical	Nonuniform slab support (variable stiffness, swelling soils, frost heave, erosion, instability, etc.) Variations in slab-based friction or bond Slab restraint Excessive panel size relative to slab thickness, foundation stiffness, slab-based friction, and applied traffic loads, and/or environmental conditions As-designed panel width (e.g., wide ramps) Inadequate saw cut depth (effective width) Too much joint reinforcement (effective width) Late sawing of joints Designs that produce excessive lateral restraint Environmental conditions (e.g., ambient temperature and moisture conditions relative to those present during placement and curing) that influence curling, warping, and drying shrinkage Stress concentrations due to embedded features (e.g., utility access blockouts) and ties to adjacent structures (e.g., transitions to different longitudinal joint patterns, adjacent structures) Construction-related aspects (e.g., timing/depth of joint sawing, timing and effectiveness of curing) Construction and service traffic loadings (load magnitude, configuration, location, number of repetitions, and strength at time of loading, etc.)
Longitudinal Cracking	Material/ Chemical	Thermal characteristics of the concrete (mainly a function of aggregate type and content) Shrinkage characteristics of the concrete (mainly a function of paste content and w/cm ratio) Concrete mixture components and proportions that affect strength development

5. Evaluation

Longitudinal cracking typically develops due to a combination of two or more of the factors listed above (e.g., nonuniform support in combination with heavy traffic loads). In addition, the timing of crack development can vary significantly, with some types of longitudinal cracking occurring very soon after construction (e.g., longitudinal cracking caused by the late sawing of longitudinal joints) while types taking months or years to develop (e.g., longitudinal cracking caused by gradual loss of uniform pavement support coupled with heavy traffic loads). Some early-age cracking may be nearly invisible or hidden by saw slurry or other construction artifacts.

This section describes various mechanisms for longitudinal cracking and notes when the associated cracking typically develops (i.e., at an early age or after a period of service and/or environmental exposure). Techniques for minimizing the potential for each cracking mechanism are also presented.

Longitudinal Cracking Caused by Excessive Slab Widths

Cause

Concrete temperature and moisture changes begin to take place as the pavement sets, hardens, and cures. The concrete responds by attempting to contract as the slab cools and dries. Slab contraction is resisted by friction or bond with the base or connections to any previously placed adjacent structures (including adjacent paving lanes and shoulders). This restraint to slab contraction produces tensile stress in the slab.

In addition, the temperature and moisture changes that develop are generally not uniform throughout the slab, often resulting in greater amounts of drying shrinkage and thermal contraction at the top of the slab than at the bottom. This causes the slab to curl (temperature-related) or warp (moisture-related) slightly in an upward direction at the panel edges and corners, resulting in reduced support, and correspondingly higher stresses, at these locations.

Two characteristics of the underlying base layer also play a significant role in the development of these critical temperature- and moisture-related slab stresses:

- **Base stiffness**: Increasing foundation layer stiffness (e.g., through the use of lean concrete, cement-treated and even asphalt-treated materials) can improve the structural support of the slab and reduce potential for foundation erosion and loss of support. However, the increased rigidity of these materials does not allow them to conform to the shape of a curled or warped slab as well as an unbound aggregate base typically does. Thus, slab support becomes more localized in the presence of temperature and moisture gradients and curling and warping stresses (as well as load-related stresses at critical locations) can increase. More information on curling and warping is provided in Chapter 10.
- **Slab-based friction or bond**: High levels of friction or bond between the concrete slab and the underlying base can restrain the contraction and shrinkage that result from decreases in slab temperature and moisture. Higher levels of tensile stress will develop as friction and/or bond strength increases. Increased levels of slab-based friction and bond also shorten the time available for sawing joints before uncontrolled cracking begins. Aggregate base layers generally provide less restraint than do most bound or stabilized base layers. Fresh concrete can penetrate permeable cement bases or asphalt-treated bases, locking the two layers together and producing a level of slab restraint that can easily lead to the development of uncontrolled cracking if a suitable joint pattern is not properly established in a timely manner.

Prevention

The magnitudes of temperature, shrinkage, curling, and warping stresses all increase with increasing panel dimensions (up to some limiting value), so panels that are too long or too wide may develop longitudinal or transverse cracks, respectively, as well as diagonal or corner cracks. Therefore, the formation or sawing of a system of joints is essential for controlling crack locations and preventing the development of excessive stresses at critical locations, especially while the concrete is young and relatively weak.

The layout and spacing of longitudinal joints in concrete pavements can influence the development of longitudinal cracking. Thicker slabs can generally accommodate larger panel dimensions (width or length), and an oft-cited rule of thumb for the maximum panel dimension is that the length or width (expressed in feet) should be no more than twice the slab thickness (expressed in inches); for example, an 8-inch thick slab should have a maximum panel dimension of 16 feet or less. However, this is a very simple guideline that does not directly account for several other critical factors, such as the stiffness of the base and the frictional characteristics of the slab-based interface. This helps to explain why some pavements that are 8 inches or more thick develop longitudinal cracks, even when the width of paving is 16 feet or less, as shown in Figures 6.2a and b.

Figure 6.2 Longitudinal cracking in pavement due to the lack of a longitudinal joint

a. Longitudinal cracking in an 8-inch pavement with a 14-foot lane width in Washington

Xu and Cebon 2017

b. Longitudinal cracking (with slot stitching) in wide ramp lane in Missouri

John Donahue, Missouri DOT

Design software such as the AASHTO Pavement ME Design Procedure (AASHTO 2015) and OptiPave 2 (TCPavements) directly consider panel size as an input into the design process and consider a range of other design and environmental factors to enable design engineers to determine suitable panel dimensions for a specific set of design conditions. Today, most concrete pavements in the U.S. are constructed with longitudinal joints that coincide with the travel lane widths (typically 12 feet apart), with some outside lanes widened to as much as 14 feet and some highway entrance/exit ramps up to 16-feet wide. When widened lanes are used, slab thickness, foundation stiffness, climate and other factors should be considered in determining whether an additional longitudinal joint should be introduced to avoid longitudinal cracking. To be effective in crack control, the joints must be sawed deeply enough so that the resulting section of concrete below the saw cut has a higher ratio of stress/strength to protect other areas from cracking. This is usually accomplished by sawing longitudinal joints to a depth of one-third the slab thickness to significantly reduce the strength of the section at that location. While unreinforced and doweled joints have historically been sawed to 1/4 the slab thickness, the presence of deformed tie bars across most sawed longitudinal joints necessitates a deeper cut (one-third the slab thickness) to offset the joint reinforcing, which might otherwise prevent the crack from forming beneath the saw cut. It is common to see longitudinal cracks form away from the sawed joint (near the ends of the tie bars) if the joint has too much reinforcing steel (i.e., too many tie bars or a "typical" number of bars in a relatively thin pavement). This underscores the need for a designed tie bar system (rather than a one-size-fits-all standard design) in new construction. This topic is discussed in more detail below.

It is noted in the previous section that base stiffness and slab-base interface friction can impact critical slab stresses that lead to longitudinal cracking. Table 6.3 presents some considerations related to the design and construction of stabilized base layers to prevent cracking.

Table 6.3 Base considerations to prevent cracking

Foundation Element	Aspect	Issue	Considerations
Base	Base Stiffness	Stiffer element base courses can lead to higher curling stresses	Limit 7-day compressive strength of lean concrete bases to 1200 lbf/in^2 or less. Use geotextile separator layer or other cushioning material between the slab and lean concrete or cement-treated base layers. Reduce panel width (e.g., half-lane width for wide exit/entrance ramps and possibly for travel lanes that are wider than 12 ft [3.7 m]).
Base	Slab–Base Friction	High friction levels or strong bond between the slab and base can restrain slab contraction and shrinkage, leading to cracking	Use debonding material between slab and bound base materials, especially lean concrete or cement-treated bases. Avoid the use of panel widths greater than 12–14 ft (3.7–4.3 m).

Longitudinal Cracking Due to Late Sawing or Inadequate Saw Cut Depth

Joint sawing is the common method of creating longitudinal joints in concrete pavement, although plastic joint inserts, tooled grooves, and other techniques have been used successfully on some projects. Regardless of the method used, the concept in each approach is the same: to create a weakened plane or section that cracks before stresses increase enough to cause cracking at other less desirable locations. The establishment of weakened plane joints in concrete pavement is a critical aspect of concrete pavement construction in terms of both depth and timing. Joints that are sawed or formed too late (after excessive stresses have already developed in the slab) or that are not created deep enough run the risk of developing random, uncontrolled cracking elsewhere in the slab. (See Figure 6.3 for an example where the quarter point joints were not sawn deep enough and the slab crack in the center of the pavement.)

Figure 6.3 Longitudinal cracking due to inadequate saw cut depths of the quarter-point longitudinal joints

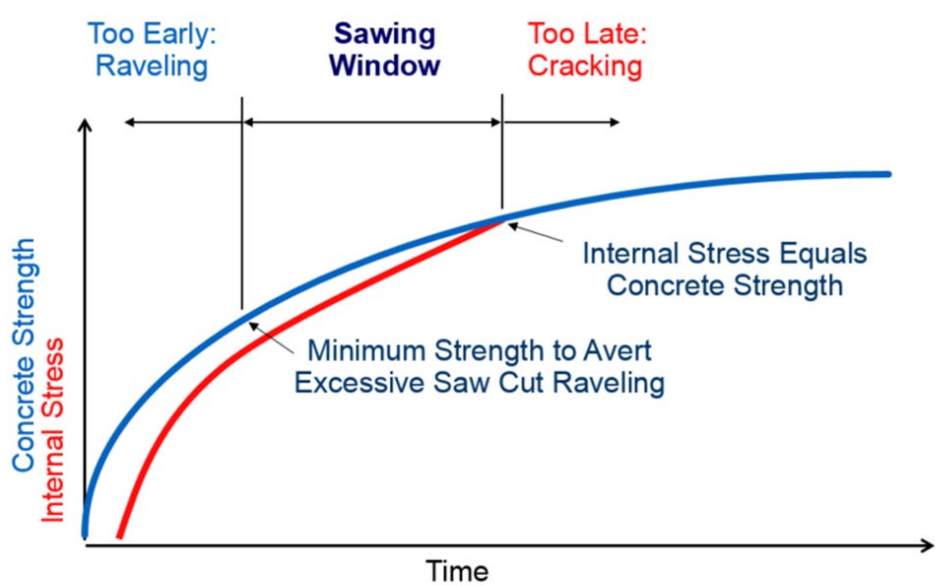
Snyder & Associates, Inc.

Cause

As noted previously, weakened-plane joints that are effective in crack control must be sawed or formed deeply enough that the resulting section of concrete below the saw cut has a higher ratio of stress/strength than any areas that are to be protected from cracking. This is usually accomplished by sawing longitudinal joints to a depth of one-third the slab thickness to significantly reduce the strength of the section at that location. While unreinforced and doweled joints have historically been sawed to one-quarter of the slab thickness, the presence of deformed tie bars across most sawed longitudinal joints necessitates a deeper cut (one-third the slab thickness) to offset the restraining effects of the joint reinforcing, which might otherwise prevent a crack from forming beneath a shallow saw cut. Note that the required depth of saw cut varies with slab thickness, which can vary within construction tolerances, between design sections, and in areas of super-elevation.

While good guidance exists on the required depth of joint creation for crack control, it can be somewhat difficult to accurately determine the proper time to cut the joint. Joint sawing should generally begin as soon as possible after the slab has achieved sufficient strength to avoid spalling or raveling of the joint during sawing. This is typically 4-to-6 hours after placement for normal concrete mixtures placed under favorable paving conditions (ACPA 1991) but can vary greatly with mixture parameters, climate conditions, curing and other factors. The time between when sawing can begin without damaging the concrete and when it will be too late to prevent uncontrolled cracking is often referred to as the “sawing window” and is illustrated conceptually in Figure 6.4.

Figure 6.4 Conceptual illustration of concrete pavement sawing window related to concrete strength and pavement stresses



American Concrete Pavement Association 2015

Figure 6.5a shows a joint that was cut to the proper depth and the crack that has developed beneath it; Figure 6.5b shows a joint cut with a crack immediately parallel to it rather than beneath the cut, most likely the result of late sawing. Figure 6.1c shows a longitudinal crack that meanders along the centerline longitudinal joint, which also is the result of late sawing or inadequate depth of cut.

Figure 6.5 Timing of sawing joints

a. Proper weakened plane saw cut and induced crack under the saw joint
Jeff Uhlmeyer Washington State DOT

b. Slab cracking occurring parallel to weakened-plane saw cut, possibly due to late sawing or inadequate saw cut depth
Derek Tompkins, American Engineering Testing, Inc.

Prevention

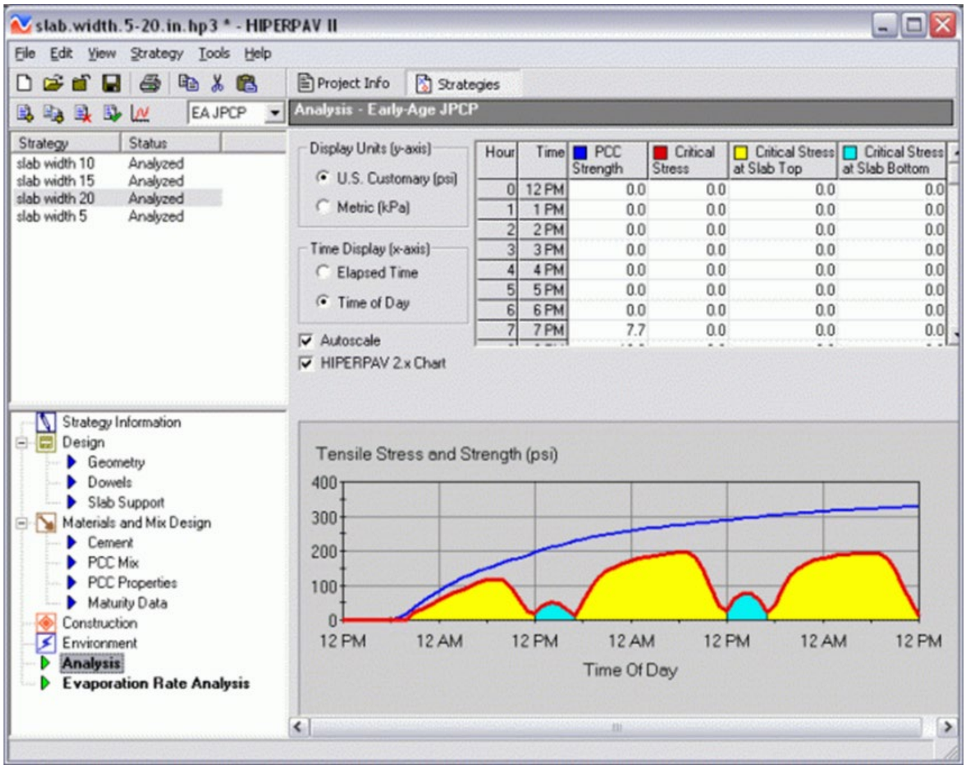
Pavement cracking due to late joint sawing and inadequate saw cut depth can be prevented by sawing the joints to the proper depth, and by performing the sawing within the sawing window.

Longitudinal joints should typically be sawed to one-third the slab thickness. Deeper cuts may be required to control cracking of highly reinforced joints (e.g., conventional pavements with closely spaced tie bars or thin pavements with tie bar sizes and quantities that are typical for thicker pavements). Saw cut depth may need to vary locally due to changes in pavement thickness (e.g., section changes, areas of super-elevation, etc.). Care must be taken to avoid sawing into reinforcing steel.

- The timing and duration of the sawing window depends on many factors, including (ACPA 2009): Concrete Mixture Parameters
    - Cement type and quantity
    - Supplementary cementitious materials – type and quantity
    - Water content
    - Use of chemical admixtures (set control, shrinkage reducing, etc.)
    - Coarse aggregate (total volume, moisture condition, thermal properties, etc.)
    - Fine aggregate (total volume, grading, use for internal curing)
    - Mix temperature at time of placement
- Pavement and Base
    - Degree of friction, bond or interlock at the slab-base interface
    - Base surface temperature and moisture condition at time of paving
    - Presence of existing pavement adjacent to new pavement
- Curing Conditions
    - Timing and effectiveness of curing techniques
    - Ambient conditions (i.e., air temperature, relative humidity, wind speed, solar radiation)
    - Sudden drops in pavement surface temperature (cold front passage or rain shower)

Experienced sawyers may determine whether the concrete can be sawed without spalling or raveling through a simple “scratch test” in which the surface is abraded with a nail or knife blade Scratch depth decreases as the concrete hardens; if the scratch removes the surface texture, then it is probably too early to saw (Taylor et al. 2006). The HIPERPAV software program (Ruiz et al. 2015) can also be used to predict when concrete stresses will exceed strength for any given concrete mixture design and construction conditions (see Figure 6.6).

Figure 6.6 Example screenshot from HIPERPAV software



It should be noted that concrete can set up at different rates through the thickness of the slab. In late season paving, for example, the bottom of the slab (resting on warmer ground) may gain strength more rapidly than the top of the slab (exposed to cool air). In this case, sawing may need to begin sooner than would be suggested by the scratch test.

As with all concrete construction, proper curing is essential to retaining moisture in the slab (thereby reducing the early development of drying shrinkage stresses) and ensuring effective strength development. This is commonly performed by applying liquid membrane-forming curing compounds on the pavement surface at rates between 100 and 200 ft^2/gal (2.45 and 4.90 m^2/L), depending on the type of pavement and the placement conditions. For example, higher rates of application may be desirable for thin concrete pavements (e.g., bonded concrete overlays) or for hot-weather paving conditions (especially with low humidity and/or higher wind speeds).

A uniformly thick application of curing compound should be placed as soon as possible after surface finishing to both the surface and sides of the slab. It may be necessary to provide more than one coat to slab sides to assure the development of an effective membrane on a vertical surface. The use of evaporation retarders (solutions that are sprayed on fresh concrete surfaces to reduce the rate of surface moisture evaporation prior to finishing) may be useful in some situations but these should not be used as finishing aids.

Longitudinal Cracking Caused by Nonuniform Support

Longitudinal cracking can be caused by localized movements, settlement, frost heave, or swelling of the base, base layers, or the natural subgrade. This may also reflect a difference in pavement support along a longitudinal boundary, as might occur when constructing a widened section of concrete pavement over the top of an older, narrower pavement without first extending the underlying pavement to the paving width. In either case, the resulting change in support, usually in combination with heavy vehicle loads, can produce a longitudinal crack that starts and ends at a longitudinal joint or panel edge and roughly traces the longitudinal boundary of the change in support along the length of the pavement (see Figure 6.7).

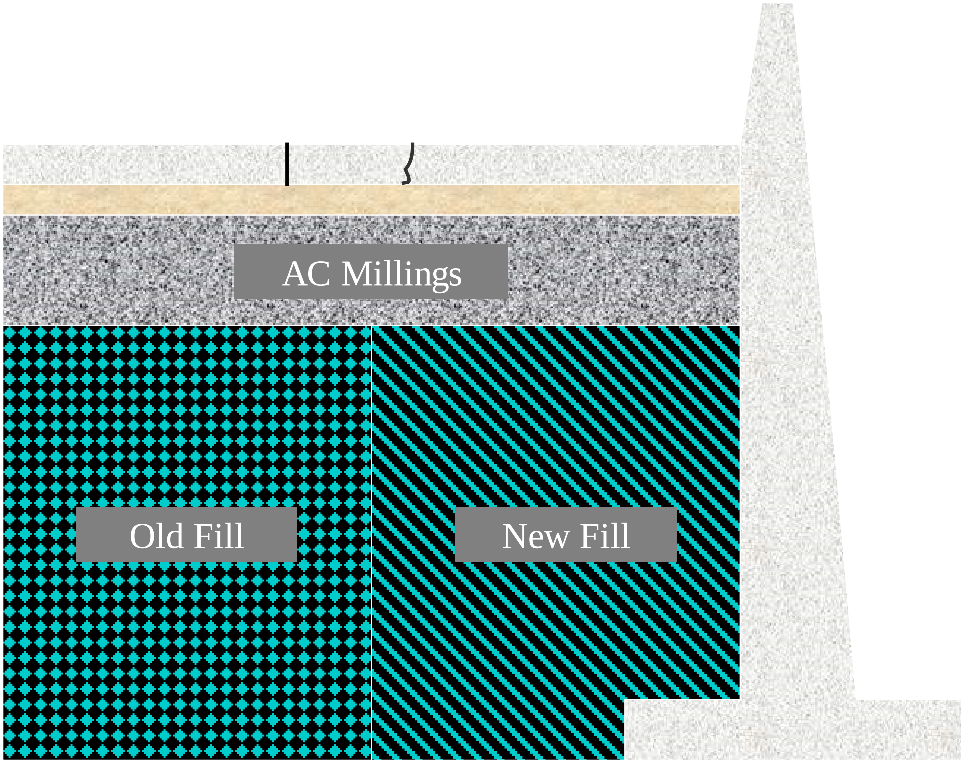
Cause

The most common source of nonuniform slab support in construction- or materials-related factors is usually related to a design factor. Each of these construction- and materials-related factors is discussed below.

Inadequate compaction: Loss of slab support may be due to post-construction settlement of the base, subbase, or roadbed soil after inadequate or variable compaction during construction. Other possible contributing factors include variability in roadbed soil materials and segregation of granular base materials. These factors can lead to locally increased slab deflections and slab stresses under traffic loads and can result in transverse, longitudinal, or diagonal cracking.

Figure 6.7 Longitudinal cracking due to longitudinally bounded variation in pavement support

a. Longitudinal cracking due to nonuniform support
John Donahue, Missouri DOT



b. Cross-section of a fill section showing different fill material resulting in differential vertical movement resulting in longitudinal pavement cracking
John Donahue, Missouri DOT

Subbase erodibility: Some subbase materials - generally granular and other unbound materials - are highly susceptible to pumping and erosion, processes in which free water in the subbase layer is transported within the subbase layer under dynamically applied heavy loads. The resulting erosion of the subbase layer (particularly the finer materials) can result in voids and areas of reduced support. Areas of the slab that overlie eroded foundation layers often experience higher deflections and stresses when subjected to heavy loads. This mechanism most commonly results in transverse and corner cracking but can result in longitudinal cracking when it takes place along a lane-shoulder joint, for example.

Subgrade volume stability: Some foundation materials (e.g., swelling/collapsing soils and frost-susceptible soils) are susceptible to volumetric changes. Soils with high plasticity indices (heavy clay) and liquid limits swell and shrink due to varying moisture content (e.g., AASHTO A-6 and A-7 classifications). Frost heave refers to the distortion or expansion of the subgrade as a result of the growth of ice lenses under freezing temperatures and requires the contribution of three factors (AASHTO 2007a):

- Sufficiently cold climate to allow freezing temperatures to penetrate the subgrade
- Supply of moisture into the freezing zone
- Susceptible soil (typically low plasticity silts which exhibit a combination of capillarity and permeability that feeds the growth of the ice lenses)

When these types of materials are uniformly distributed along a project, the entire pavement structure generally moves up or down uniformly. However, when these materials are present in local deposits, differential movements take place along the length of the pavement and may result in slab cracking. More detailed information on frost heave and swelling is found in Chapter 12.

Prevention

Table 6.4 provides a summary of general recommendations for preventing pavement settlement and support issues that might lead to the development of longitudinal slab cracking.

Note that there are competing interests in selecting a concrete pavement subbase type. For example, stabilized subbases are more resistant to erosion and loss of support than granular materials but stabilized bases are also generally stiffer than granular layers and contribute to higher curling and warping stresses and frictional forces. A compromise or balance must be struck between these considerations. Fortunately, a wide range of base materials have been used successfully beneath concrete pavements (in combination with other design features) to provide erosion-resistant and uniform support, a good construction platform and acceptable levels of curl/warp and frictional stresses.

Table 6.4 Considerations for addressing longitudinal cracking caused by nonuniform slab support.

Foundation Element	Aspect	Issue	Considerations
Base and Subgrade	Compaction /Density	Poor or inadequate compaction can lead to post-construction settlement of the base, reducing the support provided to the slab.	Ensure that all unbound foundation layers are compacted to the specified target densities.
Base and Subgrade	Uniformity of Support	Areas of nonuniformity in the foundation layers can lead to cracking	Ensure base course is homogenous and not segregated when placed. Cross-haul and mix, undercut and replace or stabilize soft spots in subgrade. Consider subgrade stabilization for plastic soils.
Base	Base Erodibility	Pumping and erosion of base beneath slab leads to unsupported conditions	Use widened lanes (if slab thickness and support conditions permit) and/or tied concrete shoulders to reduce slab deflections that induce pumping. Use bound (stabilized or treated) base materials, especially for facilities that carry heavy truck traffic. If aggregate base layers are used, limit the fines passing the No. 200 [0.075mm] sieve to 10% or less (but consider fines requirements to achieve stability during construction operations). Seal longitudinal joints (especially the lane-shoulder joint and/or provide edge drains or daylighted aggregate base to quickly remove water from the base.
Subgrade*	Swelling Soils*	Subgrade volumetric changes due to variations in subgrade moisture contents	Remove and replace small areas of swelling soils. Compact at 1–3% above optimum moisture content (AASHTO T 99). Consider use of soils stabilization and membranes.
Subgrade*	Frost Heave*	Subgrade volumetric changes due to frost penetration and growing ice lenses in subgrade	Compact slightly wet of optimum moisture content (AASHTO T 99). Use non-frost-susceptible materials within the depth of frost penetration. Protect (cover) frost-susceptible soil with sufficient thickness of non-frost-susceptible material.

*More detailed information provided in Chapter 10

The importance of drainage in helping to ensure long-term uniformity of foundation support is also emphasized. For example, isolated undercuts backfilled with granular material and pipe backfill that extends to the top of subgrade can develop into water-trapping "bathtubs." Positive drainage must be provided in these types of situations.

Longitudinal Cracking Caused by Traffic Loading

Longitudinal cracking in concrete pavements is rarely caused primarily by traffic loadings because jointed concrete pavements are typically longer than they are wide (typically 12 feet wide and 15 feet or more in length) and wheel loads tend to be channelized away from the center of the paving lane, so critical stresses tend to result in transverse cracks near mid-panel or corner cracks before longitudinal cracks develop. Load-related longitudinal cracking becomes more common in pavements that are 7 inches or less in thickness, especially as the panel lengths are reduced.

The prevention or limitation of longitudinal cracking is not design criteria in most conventional pavement thickness design approaches (e.g., AASHTOWare Pavement ME Design). It is directly considered in the design of bonded concrete overlays of asphalt pavement in both AASHTOWare Pavement ME Design and BCOA-ME (Vandenbossche 2015) where smaller panels are typically used.

There are some conditions under which traffic loads can be a primary cause of longitudinal cracking, and even more where traffic loads act in congruence with other conditions (e.g., loss of edge support) to produce longitudinal cracking. These situations are discussed below.

Cause

When panels with greater width than length are present (either by design [e.g., short lane-width repairs] or a result of a longer panel being divided by one or more transverse cracks), longitudinal cracking may develop in those panels, especially in the presence of curl/warp deformations that lift the panel edges and reduce slab support near the wheel paths. For example, Figure 6.8 is an example of a panel that may have first developed a transverse crack (resulting in the formation of two panel fragments with a greater width than length) before a longitudinal crack formed to produce a shattered slab.

Figure 6.8 Shattered or broken panel

Kim Willoughby, Washington State DOT

Half-lane-width panels with approximately equal lengths and widths are more likely to develop longitudinal cracks than transverse cracks because the wheel path crosses the transverse joints near the middle of the panel. This has been observed on some relatively thin concrete overlays with 6-by-6 feet (1.8-by-1.8 m) dimensions, as shown in Figure 6.9.

Figure 6.9 Longitudinal cracking in wheel line

John Donahue, Missouri DOT

Most often, traffic loads act accordingly with other factors to cause longitudinal cracking. Other factors may include the loss of edge support and other sources of nonuniform support, improper panel dimensions, high curl/warp stresses, etc.

Prevention

The primary methods for preventing longitudinal cracking, based solely on traffic loading, is to design and construct pavements with the correct thickness, consider panel dimensions, and aspect ratios. Properly designed and constructed conventional jointed concrete pavements (i.e., typical 12-by-15-foot panel sizes with doweled joints and uniform foundation support) will develop transverse cracks before longitudinal cracks. AASHTOWare Pavement ME Design and other current design procedures use transverse cracking as a performance criterion in design. In these cases, traffic-related longitudinal cracking should never be a concern. The prevention of other modes of longitudinal cracking (for which traffic loading can play a contributing role) are discussed in other sections of this chapter.

Longitudinal Cracking at Intersections

Urban roadway environments present significant jointing challenges because of the presence of intersecting roadways, turning lane adds and drops, median islands, curb/gutter installations and in-pavement utilities. The jointing patterns must be carefully planned to avoid random cracking that may detract from the performance and/or function of the pavement.

It can be noted that, for intersection paving, jointing and cracking orientation within the intersection is relative to the direction of travel. A joint or crack is longitudinal to vehicles traveling in a perpendicular direction.

Cause

The primary cause for transverse and diagonal cracking in urban concrete intersections is related to the layout of the joints. While most agencies work to maintain the same maximum joint spacing used in their conventional concrete paving, some variance in that spacing is often needed to avoid issues with matching joint patterns in existing pavement structures, curbs, and adjacent traffic lanes. Furthermore, the layout of joints at sharp, acute angles to one another, mismatching joints and failing to produce panels that are roughly square in shape can lead to local stress concentrations and uncontrolled slab cracking.

Prevention

General recommendations for addressing unique jointing situations in urban environments include (ACPA 2007b):

- Prepare a jointing plan.
- Match and align joints.
- Avoid joint intersection angles less than 60º.
- Keep slabs short (~12 feet) and relatively square.
- Avoid odd-shaped slabs.
- Accommodate in-pavement structures through jointing and boxouts.
- Make field adjustments as needed.

A maximum joint spacing should be selected with consideration of the nominal slab thickness and base conditions (stiffness and friction/bond) and this spacing should be used in the general layout of the intersection joints. It is common to carry the traffic lane widths from both directions through the intersection for simple right-angle intersections, typically resulting in nominal 12-by-12 foot panels in the intersection core.

Some adjustments to those dimensions may be needed to help match existing pavement joints and to accommodate in-pavement structures such as manholes, utility valves, and drainage inlets. These in-place structures will often require boxouts with a perimeter isolation joint to accommodate movements, and the boxouts themselves will need to be carefully jointed to avoid initiating cracks from stress concentrations at boxout corners. In lieu of boxouts, some contractors elect to wrap the in-place structure with a pliable isolation joint filler or may cast a telescoping structure top directly into the concrete. ACPA (2007b) provides a 10-step process for effectively laying out intersection jointing in a manner that minimizes potential for cracking.

Longitudinal Sympathy Cracks

"Sympathy cracking" is the term used to describe cracks that propagate from abutting joints in adjacent paving lanes or structures. Sympathy cracks are commonly observed in urban streets and intersections where mismatched or T joints occur more frequently, and most sympathy cracks tend to be transversely oriented because it is relatively uncommon to see mismatched longitudinal joints (since they tend to fall along common lane lies). Photos and descriptions of transverse sympathy cracks are presented in Chapter 5.

Cause

When a concrete pavement joint (or crack) terminates up against the unjointed/uncracked edge of another panel, that joint or crack may open and close in response to temperature changes. The movement of that joint or crack will be somewhat restrained by friction or mechanical connections (i.e., dowel or tie bars) with the adjacent panel, which creates a tensile stress concentration at the face of the uncracked. When the restraining stress exceeds the strength of the uncracked slab (or when it develops enough fatigue from repeated stress cycles), a crack will begin to propagate across the slab. The timing of the occurrence of these cracks can vary depending on the environmental conditions, panel lengths, base type and friction, and the degree of friction or restraint provided at the joint intersection.

Prevention

There are several approaches used to minimize the potential for developing any type of sympathy cracking. Approaches include the following.

- Lay out all joints to avoid T joint intersections.
- Match joints exactly and make the top of the T an isolation joint for at least a few feet (~1 m) on either side of the stem of the T to reduce the stress concentration.
- Drill, core, or form a hole through the full thickness of the pavement at the joint intersection.
- Use diagonal joints to transition from one joint pattern to another (see Figure 6.10).
- When the location of the T joint can be determined prior to placement of the concrete, place supplemental reinforcing bars just above the top of the T to reduce the potential for a crack to migrate into the slab beyond the bars. The use of two lengths of No. 4 (1/2-inch) deformed bar approximately 3 feet in length have been successfully used when placed at least 2 inches below the pavement surface at distances of 3 and 6 inches from the joint face.
- The use of steel or synthetic fiber-reinforced concrete may resist sympathy cracking at a higher pavement cost.

Figure 6.10 Transition between varying longitudinal joint patterns using core hole and diagonal joints to avoid sympathy cracking

Mark Snyder, PERC

Other Longitudinal Cracking Mechanisms

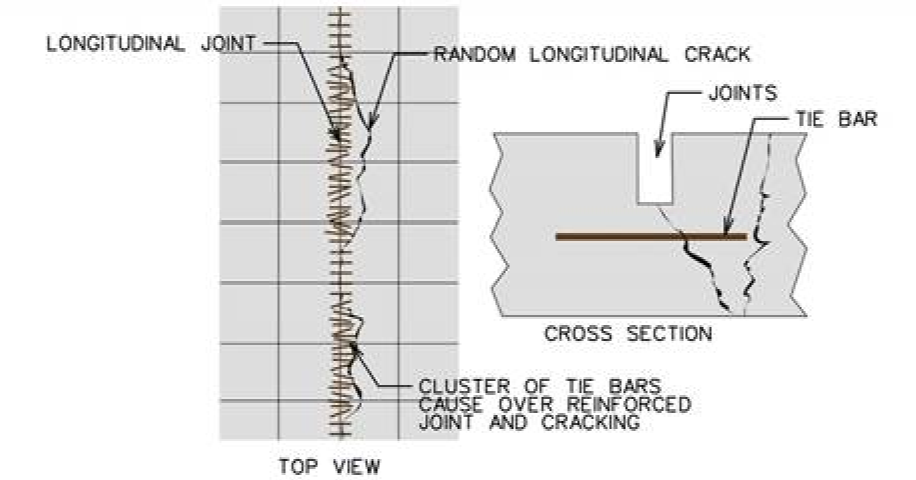
This section describes and discusses a few less common longitudinal cracking mechanisms, which are included below.

- Excessive reinforcing of tied longitudinal joints (i.e., too many tie bars)
- Construction timing in tying new concrete lanes to existing lanes with activated transverse joints (especially dominant joints)
- Tendon failures in post-tensioned concrete pavement

Over-Reinforcing of Tied Longitudinal Joints: The inclusion of too many (cluster; see Figure 6.11) or too large of tie bars can reinforced the joint and offset the effects of joint sawing to create a weakened plane for joint formation. In such cases, a crack often forms 18 to 24 inches from the joint, just beyond the ends of the tie bars (see Figure 6.11).

This type of design sometimes results from the use of a standard one-size-fits-all tie bar design, without regard for panel thickness, friction, or bond between the slab and base, or the distance the nearest free edge.

Figure 6.11 Longitudinal cracking along the ends of tie bars on a 7-inch concrete containing tie bars that were clustered together resulting in reinforcing the joint



Snyder & Associates, Inc.

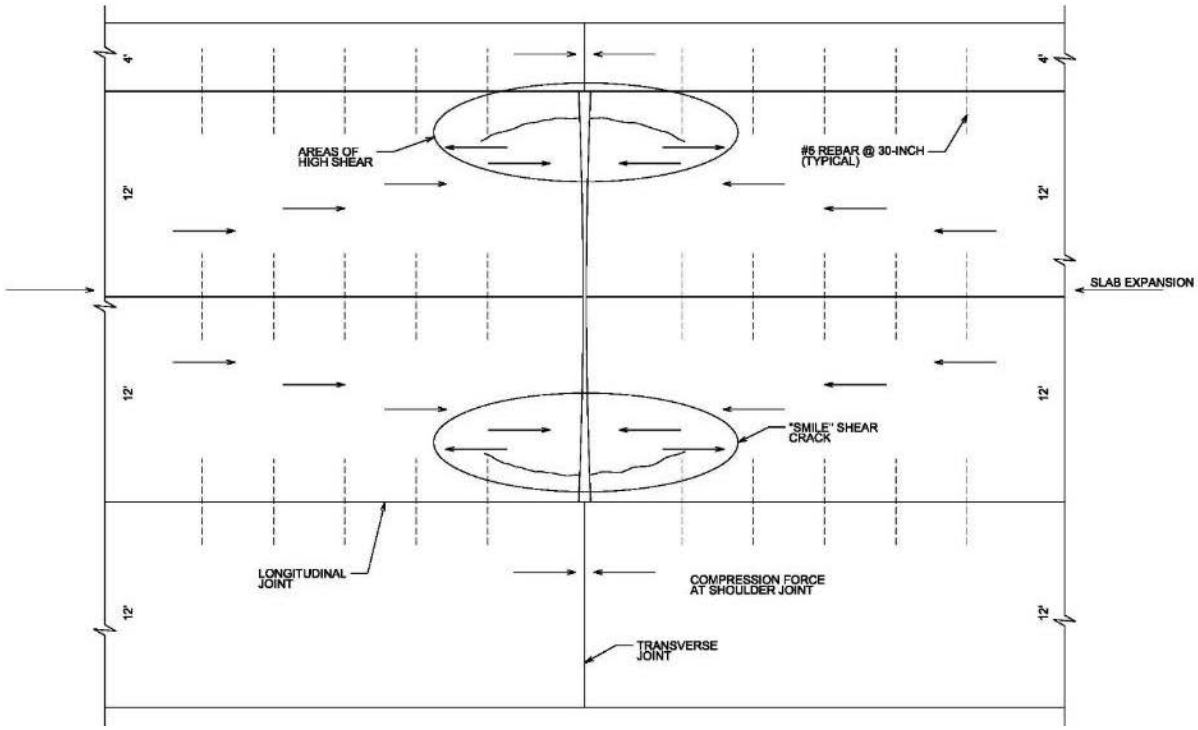
This type of cracking can usually be avoided by performing a proper tie bar design (one that considers the primary factors that should influence tie bar design at any given longitudinal joint). A simple but generally effective tie bar design procedure based on "subgrade drag theory" is presented in equation and nomograph form in the 1993 *AASHTO Guide for Design of Pavement Structures*. A more comprehensive mechanistic-empirical approach to tie bar design is presented in Mallela et al (2009).

Tying New Lanes to Existing Lanes with Activated Transverse Joints: Longitudinal cracking has been associated with excessive restraint of longitudinal temperature-related expansion in the mainline pavement due to construction of a tied shoulder pavement well after the construction of the mainline pavement, often during cooler weather.

When the mainline pavement is constructed first during warmer months, it can experience both drying shrinkage and thermal contraction before the shoulder pavement is placed, resulting in the placement of a continuous concrete shoulder against a series of mainline pavement joints that have activated and opened to some width. In many cases, only some of the mainline pavement joints will have activated a short time after placement, creating what are sometimes called "dominant joints" at every third, fourth, or fifth joint. These dominant joints open sometimes three, four, or five times as wide as would be observed if all the joints were active.

In warmer weather following construction, thermal expansion drives all the joints towards a fully closed condition. Dominant mainline pavement joints are unable to close all the way because the adjacent transverse joints in the tied shoulder close first, thereby restraining the mainline pavement joints from closing completely and causing very large shear stresses to build along the tied longitudinal joint interface (see Figure 6.12).

Figure 6.12 Longitudinal cracking in existing pavement with activated joints when joint movement is restrained by tying on a new adjacent lane



National Concrete Pavement Technology Center

If the shoulder concrete is of similar strength to that of the mainline pavement, a shear fracture in the mainline pavement initiates at the transverse joint, often directly over the dowel bar. Dowel bars reduce the effective thickness of the overlay at that location, creating a plane of weakness as seen in Figure 6.13.

Figure 6.13 Longitudinal shear restraint crack of dowel initiating over dowels at two locations on opposite sides of the joint

John Becker, American Concrete Pavement Association Pennsylvania Chapter

If the shoulder section was weaker than the mainline pavement (e.g., due to the use of a thinner section or less material strength at the time of expansion), cracking or crushing of the shoulder concrete may occur instead. This mechanism may be exacerbated by the over-use of tie bars in the longitudinal lane-shoulder joint.

This mechanism may be mitigated by: 1) paving adjacent lanes when all existing lane joints are uniformly activated and of similar narrow width (e.g., paving in warmer weather); 2) plunge-cutting new pavement transverse joints to provide a width that is similar to that of the existing pavement joints (to eliminate restraint of existing pavement joint closure in warmer weather); and/or 3) using no more tie bars than analyses indicate is necessary and providing as much distance between the joint and the first tie bar on either side as possible. The use of isolation or bond-breaking material along the joint between the tie bars may also be effective in reducing stress concentrations along the joint.

Tendon Failures in Post-Tensioned Concrete Pavement: Post-tensioned concrete pavement (both cast-in-place and, more recently, precast) has been used on some highway and airfield paving projects throughout the U.S. Post-tensioned (prestressed) concrete offers the potential to construct pavement in lengths of several hundred feet without working transverse joints and their associated maintenance issues. It also allows the use of thinner pavement sections to carry the same loads as conventional jointed concrete pavement. Both of these potential benefits is achieved by using steel post-tensioning strands in embedded ducts to place the hardened concrete in compression, thereby offsetting critical load-related tensile stresses.

If one or more of the post-tensioning strands fails, the offsetting compression in the surrounding concrete is reduced and effective tensile forces increase. In addition, a shear field develops between the highly stressed concrete surrounding adjacent intact tendons and the less-stress concrete surrounding the failed tendon(s). The result of either (or both) of these mechanisms is often a longitudinal crack directly over the failed tendon that runs the entire length of the prestressed unit (see Figure 6.14).

Figure 6.14 Longitudinal crack in over-failed strand in post-tensioned concrete pavement on Pennsylvania I-99

Mark Snyder, PERC

This type of failure is difficult to prevent by any means other than the use of good prestressed pavement design and construction techniques, including the grouting of tendon ducts after tensioning to reduce the chance of corrosion-induced tendon failure.

Table 6.5 summarizes the key points related to the causes and prevention of longitudinal cracking in concrete pavements.

Table 6.5 Overall summary of causes and prevention of longitudinal cracking

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Excessive Panel Width Cracking	Greater slab widths increase critical curling, warping, and shrinkage stresses in the slab that can lead to cracking Exacerbated by increased base stiffness and friction or bond	Employ suitable panel width for climate conditions, foundation support and friction, and slab thickness Avoid over-tying longitudinal joints to create a wider effective panel width	Avoid high-shrinkage mixtures (high water and paste contents, high CTE aggregates). Minimize mixture paste content Avoid using exceedingly stiff base Use interlayer between slab and stabilized base	Saw joints deeply enough to ensure joint activation
Late Sawing or inadequate Saw cut depth	Inadequate saw cut depth Late sawing		Maximize sawing window through good materials selection and mixture proportioning, good curing materials	Maximize sawing window with proper coverage and timely application of curing techniques and site control techniques (e.g., fogging, shading and wind breaks where needed and feasible) Saw joints within the "window of opportunity" Saw joints to the specified depth Monitor early-age strength development Employ HIPERPAV or other software to determine cracking risk for potential paving scenarios

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Nonuniform Support Damage	Inadequate compaction of foundation materials Erosion of foundation materials Volumetric changes in foundation materials (swelling, frost heave, etc.)	Install pavement drains to reduce moisture in foundation materials Consider use of widened lanes (if slab thickness and support conditions permit) and/or tied concrete shoulders to reduce slab deflections that induce pumping. Seal longitudinal joints	Ensure homogeneous foundation layers Consider stabilized or treated base materials Consider stabilization of plastic subgrade soils Limit fines in unbound base layers Remove and replace areas of swelling soils	Compact all foundation layers to target densities Compact swelling soils and frost-susceptible soils slightly wet of optimum moisture content
Traffic Loading Stress Damage	Excessive stresses in the slab due to truck loading combined with Short and wide panels or half-lane-width panels	Design and construct pavements with appropriate slab thickness and panel dimensions for traffic levels, climate conditions, foundation support and friction	N/A	Construct pavement to proper thickness tolerances

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Intersection Cracking	Poor layout of joints Failure to match adjacent joints Use of acute joint angles Joints terminating against adjacent slabs Odd-shaped slabs Utilities not jointed properly	Develop appropriate jointing plan for project scenario (including the accommodation of in-pavement structures and utilities) Keep joints short and relatively square Use ACPA 10-step process (or similar) for intersection joint layout	N/A	Follow jointing plan making field modifications as appropriate. Match and align joints wherever possible
Sympathy Cracking	Stress concentration in slabs caused by movement of terminating joints that open and close (due to temperature changes) against adjacent pavement	Lay out joints to avoid use of T joint intersections Accommodate in-pavement structures and utilities Isolate new slab with isolation joint Use diagonal joints to transition between joint patterns	Use fiber-reinforced concrete to resist sympathy cracking	Core joint intersections to full depth to reduce stress concentration Place supplemental reinforcing steel to stop propagation of sympathy cracking
Over-Reinforcing Tied Longitudinal Joints	Too many (or too large) tie bars offset effects of joint sawing and prevent joint activation	Properly design tie bar systems for each longitudinal joint on each project	N/A	Saw longitudinal joints to the proper depth during the sawing window

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Tying New Lanes to Existing Lanes with Activated Transverse Joints Damage	New lanes placed adjacent and tied to existing pavement with active dominant joint restrain eventual closure of those joints, joints causing shear cracks to develop in the weaker lane	Properly design tie bar systems for each longitudinal joint Consider increasing distance from the transverse joints to the first tie bar on either side	N/A	Construct adjacent lanes when existing lane joints are all tight If the above is not possible, plunge-cut adjacent transverse joints to provide full-depth joint width that matches or exceeds width in existing pavement
Post-Tensioned Concrete Failure	Failure of post-tensioning strand induces localized shear stresses and reduced offset of tensile stress in the surrounding concrete		Use corrosion-resistant or coated tensioning strand	Grout post-tensioning ducts to prevent strand corrosion due to moisture intrusion

6. Treatment and Repairs

General repair methods and maintenance approaches to address longitudinal cracking distress in concrete pavements are described in the following section. It should be noted that, in some cases, the cracking distress does not pose a significant performance issue and may be best left untreated (for example, if the cracking is tight and/or if the pavement is exposed to low truck traffic levels).

Repairs

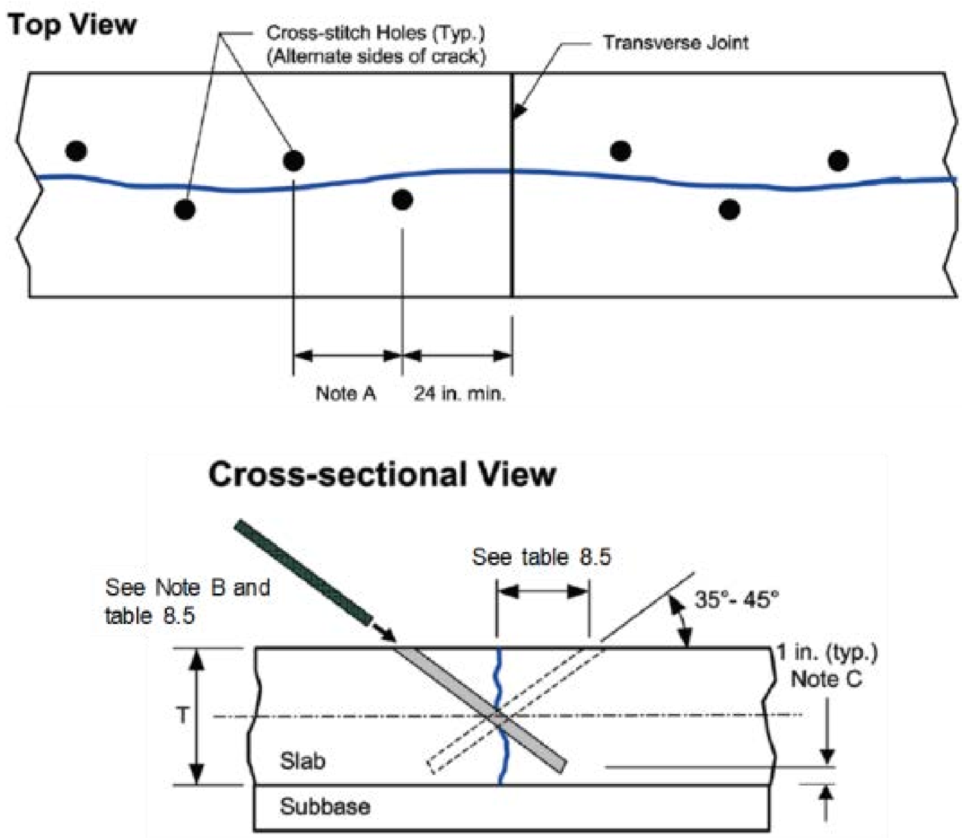
Full-Depth Repair

Full-depth repair and slab replacement are commonly used to address deteriorated longitudinal cracks in concrete pavements. These repairs consist of the removal of isolated deteriorated areas through the entire thickness of the existing slab and replacement with a concrete replacement material. The *Concrete Pavement Preservation Guide* (Smith et al. 2014) provides details on the design and construction of effective full-depth repairs and slab replacements. Subgrade support and drainage issues should be addressed concurrently.

Cross-Stitching and Slot Stitching

Cross-stitching and slot stitching are alternatives to full-depth repair or slab replacement when addressing longitudinal cracks. These techniques install deformed tied bars either in holes that are drilled at angles through the crack from both sides (Figure 6.15) or in slots across the crack (Figure 6.16).

Figure 6.15 Cross-stitching



a. Schematic of cross-stitching

Smith et al. 2014

A: Distance between holes varies based on truck traffic levels; 20-inch spacings recommended for heavy traffic and 30-inch spacings for light traffic.

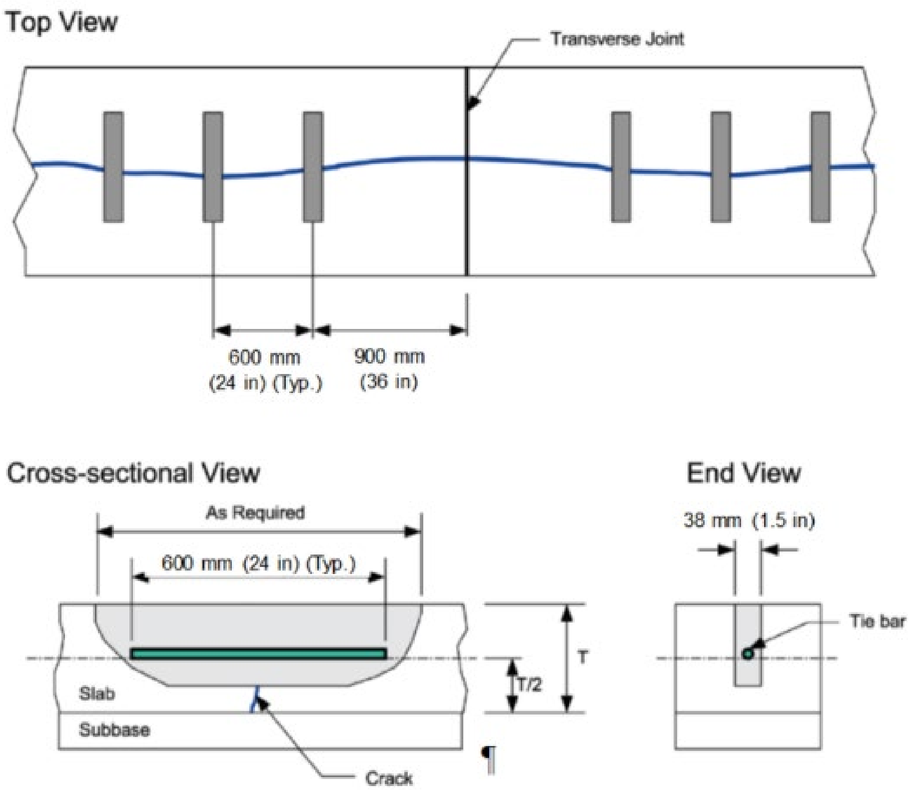
B: Epoxy deformed bar into hole. Provide 1-inch cover at bottom of the hole.

C: Do not drill hole completely through slab. Stop drilling so that the epoxy/grout will not run out of the bottom while backfilling.

b. Cross-stitched longitudinal crack

Smith et al. 2014

Figure 6.16 Slot stitching



a. Schematic of slot stitching

Smith et al. 2014

b. Slot-stitched longitudinal crack

John Donahue, Missouri DOT

These bars help to hold the crack tight and maintain good load transfer across the crack through aggregate interlock, thereby reducing deflections, pumping, and further crack deterioration. Crack sealing is often performed in conjunction with cross-stitching and slot stitching if the crack is not tight at the time of repair. Diamond grinding may also be performed in conjunction with these techniques to restore pavement surface profile.

These techniques can prevent further crack opening and help to maintain aggregate interlock load transfer but typically will not improve load transfer unless dowel-sized deformed bars are used and spaced closely (e.g., 12-inch centers). More details on the use of cross-stitching and slot stitching is available in the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

Crack Sealing or Filling

Crack sealing or filling involves placing a sealant material in the crack to prevent the intrusion of water or incompressibles that could accelerate deterioration of the crack and surrounding pavement structure. Depending on the width of the crack, it could involve establishing a sealant reservoir using a small-diameter diamond saw or a router bit to "chase" the crack, followed by filling the crack with a liquid sealant material (joint filling) or inserting a backer rod prior to sealant installation to establish a designed shape factor (joint sealing). Tight, nonworking cracks are typically left unsealed.

More details on the use of cross-stitching and slot stitching are available in the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

Diamond Grinding

According to the *Concrete Pavement Preservation Guide* (Smith et al. 2014), diamond grinding involves the removal of a thin layer of concrete (typically 0.12 to 0.25 inches) from the placement surface using a self-propelled machine outfitted with a series of closely spaced diamond saw blades mounted on a rotating shaft. Diamond grinding does not directly address slab cracking but may be used in conjunction with the treatments listed above to eliminate profile issues across a repaired crack and restore overall pavement rideability.

Maintenance

Crack Sealing

Crack sealing consists of sawing, power cleaning, and sealing cracks that are wider than 0.125 inches using high-quality sealant materials. Sealing reduces the amount of water that is allowed to infiltrate the pavement and prevents incompressible materials from intrusion (Smith et al. 2014). Consequently, this may help prevent or minimize the further breakdown and deterioration of the crack; the operation may also help to reduce the potential for spalling. Crack sealing may be performed on cracks up to about 0.5 inches wide but it is generally recommended that tight, low-severity cracks be left unsealed. Overall, this may be a short-term solution to minimizing water infiltration and does nothing directly to address the deterioration that has already occurred.

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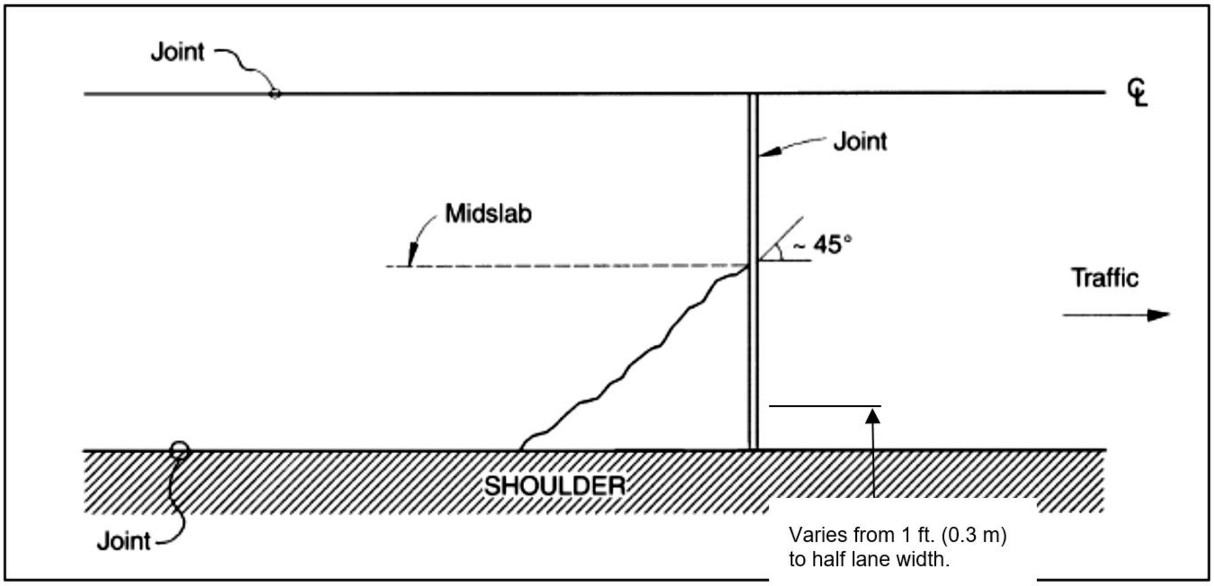
CHAPTER 7. CORNER CRACKING

1. Description

Corner cracking (also known as a “corner break”) is a distinct full-depth fracture in a jointed concrete pavement. Corner cracks intersect adjacent transverse and longitudinal joints at an angle of approximately 45 degrees with the direction of traffic. The lengths of the sides are rarely less than 1 foot (0.3 m) and are always less than half the width of the slab (by definition) on each side of the corner. Cracks with longer legs are considered diagonal cracks (see Chapter 5). Figure 7.1 presents a dimensioned illustration of a corner crack and Figure 7.2 presents some typical photos of corner cracking.

Figure 7.1 Illustration of a corner crack



Miller and Bellinger 2014

Figure 7.2 Corner cracks in concrete pavements

a. Low-severity interior corner crack
Mark Snyder, PERC

b. Moderate-severity corner crack
Miller and Bellinger 2014

c. High-severity interior corner breaks
Peter Tayor, National Concrete Pavement Technology Center

d. Medium-severity corner crack
Peter Tayor, National Concrete Pavement Technology Center

2. Severity

The *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) lists corner breaks (cracking) as a distress for jointed concrete pavements (JCPs). Table 7.1 summarizes the LTPP severity levels and measurement methods for corner breaks.

Table 7.1 Severity levels and measurement

Distress	Description and Severity Levels	Measurement
Corner Cracking - JCP	Cracks that intersect adjacent transverse and longitudinal joints at an angle of approximately 45 degrees with the direction of traffic. The lengths of the sides are from 1 ft (0.3 m) to half the width of the slab on each side of the corner. **Low**: The crack is not spalled for more than 10% of the length of the crack. There is no measurable faulting, and the corner piece is not broken into two or more pieces and has no loss of material and no patching. **Medium**: The crack is spalled at low severity for more than 10% of its total length or faulting of the crack or joint is less than 1/2 in. (13 mm) and the corner piece is not broken into two or more pieces. **High**: The crack is spalled at moderate-to-high severity for more than 10% of its total length or faulting of the crack or joint is greater than 1/2 in. (13 mm), or the corner piece is broken into two or more pieces or contains patch material.	Record the number of corner breaks at each severity level.

Source: Modified from Miller and Bellinger 2014

3. Testing

Field and laboratory tests are described below. For further detail and additional information, refer to Chapter 19.

Field Tests

Following the pavement distress field survey in which corner cracking is identified and recorded, some follow-up field testing may be needed to help determine possible causes of the distress, as well as to assist in the identification of appropriate preservation or rehabilitation solutions. These additional field tests include the following.

Coring

Concrete core samples can be retrieved at the cracks to determine the depth of the crack penetration through the slab, as well as to verify the thickness of the slab. This also provides the opportunity to observe the tortuosity of the cracking through the concrete and determine whether the crack proceeds around or through the coarse aggregate particles. Cracking around an aggregate particle suggests an early cracking mechanism, while cracking through the aggregate (when paste bond strength has increased) suggests later age cracking (Walker, Lane, and Stutzman 2006), although these guidelines are not absolute.

Coring directly over and through any dowels at the transverse joint (particularly dowels located closest to the nearest longitudinal joint) will allow inspection of the condition of the dowels and the surrounding concrete at the joint face. Badly corroded dowels will experience a loss of cross-section that makes them less stiff and results in higher relative deflections across a joint of any given width. Failure of the concrete surrounding the dowel may result in either support gaps above and below the dowel or spalling of the concrete around the dowel at the joint face; either condition results in an effective increase in joint width and higher relative deflections across the joint.

Coring can also be used to assess the condition and potential bond or friction characteristics of the underlying base below the crack and determine whether loss of slab support developed due to erosion at the slab-to-base interface or in a lower layer.

Straightedge or String Line Testing

A straightedge can provide an indication of corner crack depth when the cracking is caused (at least in part) by a void or other loss of support. The straightedge should be rigid and sufficiently long to be able to establish the general slope of the intact portion of the slab and to project far enough across the cracked portion of the slab to reveal a break in the slopes of the two slab fragments at the crack. A break in slope may also be visible by anchoring a string line or cable to the surface of the uncracked portion of the slab several feet from the crack and pulling the string line taut along the surface and over the cracked portion of the slab. If there is no apparent break in the slope, the corner break may not yet extend through the full slab thickness where a void is present or is not due to a loss of support. Other types of testing (described below) can provide an indication of whether a void or other loss of support exists below a pavement corner

Falling Weight Deflectometer (FWD) Testing

Deflection testing can be performed (using an FWD or other devices) prior to the development of full-depth corner cracking to evaluate the underlying support conditions and detect voids. After slab corners crack completely, FWD results become less meaningful and more difficult to interpret. FWDs can be used to assess the load transfer characteristics of the cracks.

Drainage Survey

If a pavement drainage system is present, an inspection should be conducted to determine whether it is functioning properly. At a minimum, this inspection should include locating and inspecting drainage outlets to ensure that they are properly located at low spots in the longitudinal pavement profile and are not crushed, submerged, or otherwise impede the outflow of pavement moisture. A more sophisticated inspection could include the use of video camera systems (inserted through the surface or through drainage outlets). A functional test of the drainage system could include the discharge of water trucks on the pavement surface and monitoring outlets to check their function and drain time.

Guidance on the conduct of drainage surveys can be found in Smith et al. (2014) and Darter et al. (1998).

Ground-Penetrating Radar (GPR) and Ultrasonic Echo Imaging

GPR and ultrasonic testing devices may be useful in identifying the presence and size or extent of voids beneath low-severity corner cracks that have not yet fully broken and settled.

Laboratory Tests

Laboratory tests of field-drilled concrete samples typically provide little useful information in forensic evaluations of corner cracking. Gradation and Atterberg limit testing of aggregate base and subgrade samples obtained through core holes may be useful in identifying unbound material characteristics (or changes in characteristics since construction) that have contributed to loss of pavement support.

4. Identification of Causes

There are relatively few mechanisms associated with the development of corner cracking. One involves top-down cracking due to wheel load applications at corners that have poor foundation support and/or poor edge support. The other involves the development of high shear stresses near the longitudinal joint when transverse joints cannot close in warm weather because of infiltration of concrete mortar during the construction of adjacent lanes or shoulders during cooler weather. Table 7.2 provides a summary of the factors that contribute to these corner cracking mechanisms.

Table 7.2 Summary of causes and contributing factors associated with corner cracking of jointed concrete pavements

Distress	Category	Description of Causes and Contributing Factors
Corner Cracking (all JCP types)	Mechanical	Heavy loads in wheel paths and slab corners, generally in combination with one or more of the following contributing factors: Poor transverse joint load transfer (i.e., ineffective or missing transverse joint dowels, ineffective aggregate interlock, etc.) Poor longitudinal edge support (i.e., no widened late or tied concrete shoulder, missing lane ties, etc.) Loss of foundation support due to pumping, erosion, slab curl, or other mechanisms Nonuniform slab support due to inadequate controls on subgrade and base during original construction Volume change of subgrade soils (shrinkage, swelling and frost heave) Interference between joint ties and dowels that results in restraint of slab thermal and moisture movements in corners
Corner Cracking (all JCP types)	Construction	Inappropriate jointing practices that result in acutely angled panel corners, especially for longer panels where curl/warp stresses are greater For placement of new concrete adjacent to existing pavement (e.g., adding shoulders to existing concrete pavement or construction of adjacent travel lanes in different paving seasons): Presence of relatively wide joints in existing pavement when constructing adjacent lanes or shoulder Failure to caulk or otherwise seal the vertical exposure of these joints in the longitudinal slab edges prior to placing the adjacent pavement Penetration of plastic concrete mortar into open joints during construction Restraint of joint closure in warmer weather by infilled mortar

5. Evaluation

Corner cracking in concrete pavements typically develops due to repeated applications of heavy wheel loads near panel corners that exhibit high deflections due to poor foundation support and/or poor support of the transverse and longitudinal panel edges. Less frequently, corner cracking develops due to localized restraint of transverse joint closure due to infiltration of mortar or other incompressible materials into the transverse joint at the longitudinal joint during placement of an adjacent lane or shoulder. The timing of corner crack development and severity of presentation can vary significantly and depend on many factors.

This section describes primary mechanisms for corner cracking and notes when the associated cracking typically develops (i.e., at an early age or after a period of service and/or environmental exposure). Techniques for minimizing the potential for each cracking mechanism are also presented.

Corner Cracking Caused by Corner Loading with Inadequate Slab Support

The most common type of concrete pavement corner cracking is a top-down crack that develops due to repeated heavy loads passing over a slab corner that is poorly supported. Poor support of the slab corner could result from one or more of several mechanisms (e.g., unstable soils, poor initial compaction, etc.) but is typically a result of pumping and base/subgrade erosion.

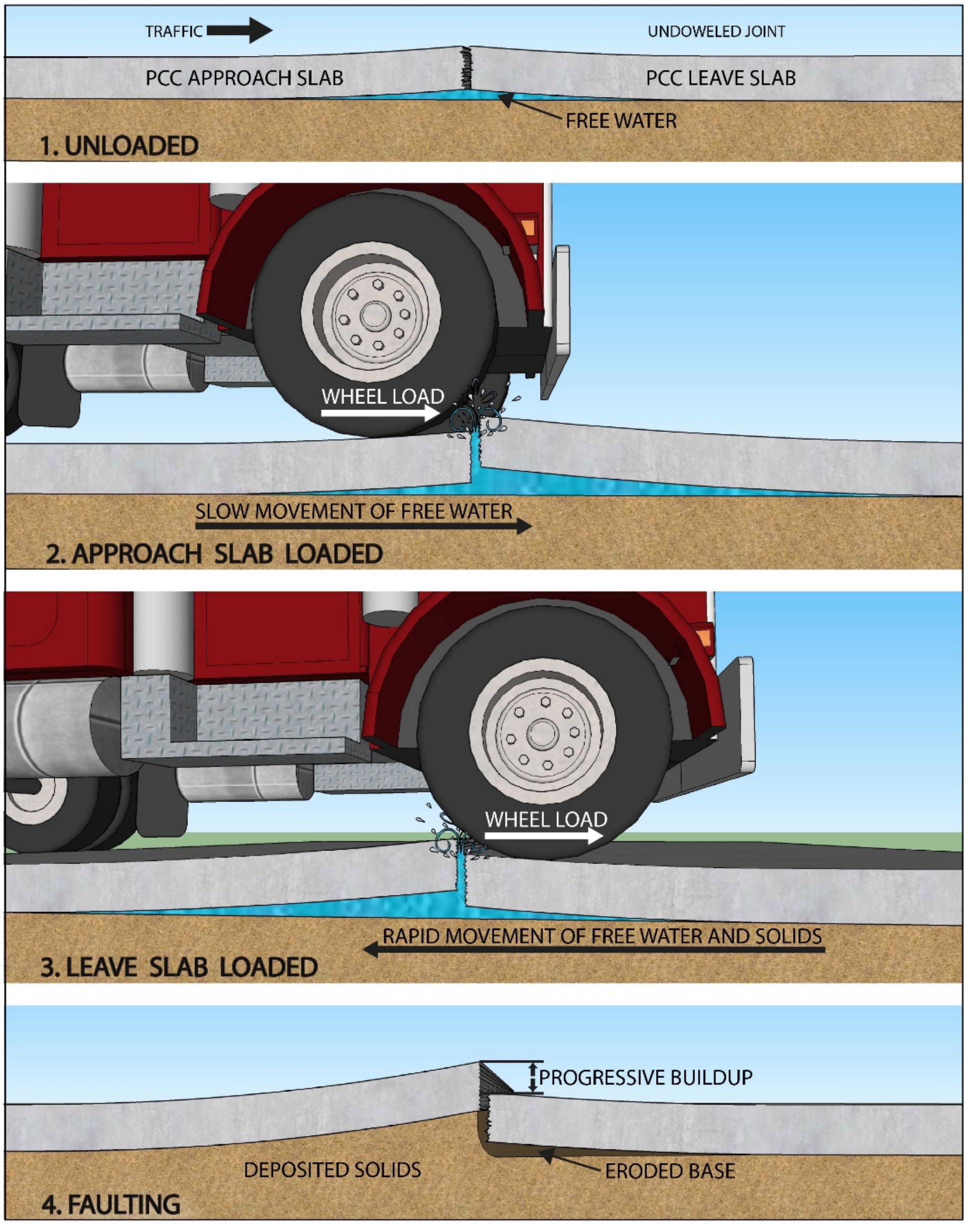
The development of pumping and associated base/subgrade erosion requires four conditions:

- An erodible base/subgrade material
- Free water in the pavement structure (usually due to surface infiltration through unsealed joints and crack, along with inadequate drainage to remove the water)
- Poor mechanical load transfer across the transverse joint, the longitudinal joint, or both (i.e., high potential for differential movement across the joints when the slab it loaded)
- Heavy dynamic loads to initiate differential movement across the joints

When any one (or more) of these four conditions is removed, pumping and associated erosion cannot develop, and corner cracking is highly unlikely.

When all four conditions are present, the erosion mechanism begins as the wheel load approaches the transverse joint and moves water along a slab/base/subgrade interface toward the joint. As the wheel load crosses the joint, inadequate load transfer allows the approach side of the joint to rebound while the leave side of the joint is suddenly deflected, rapidly ejecting water back under the approach side (and sometimes vertically through the joint—see Figures 7.3 and 7.4).

Figure 7.3 Schematic of a pumping mechanism



Snyder & Associates, Inc.

Figure 7.4 Water that has been ejected from a transverse joint under a moving wheel load

Tom Burnham, Minnesota DOT

Water ejected vertically through the joint sometimes carries fine material eroded from the base or subgrade, which is deposited on the shoulder (see Figure 7.5) and may be a visible indicator of lost support.

Figure 7.5 Pumped foundation materials deposited on a shoulder and localized collapse of asphalt shoulder adjacent to transverse pavement joints

Jeff Uhlmeyer, Washington State DOT

The repeated dynamic movement of water has the potential to erode all but the most well-stabilized foundation materials on either side of the joint (but especially on the leave side), leaving a void that results in higher pavement corner deflections and stresses. With repeated heavy load applications, these higher deflections and stresses often result in the development of a corner crack (break). This same mechanism may produce joint faulting before the development of corner cracking (see Chapter 9).

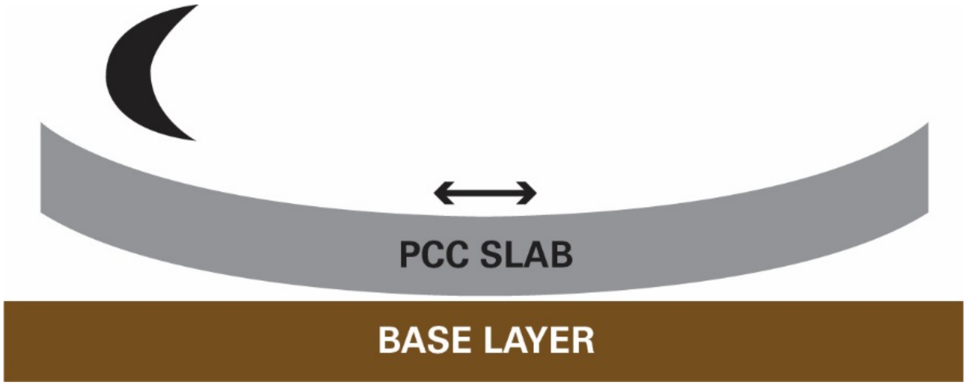
A similar mechanism can take place when heavy loads ride on an asphalt or untied concrete shoulder, causing the shoulder to deflect relative to the mainline pavement and moving water across and along the lane-shoulder joint, causing loss of support which can result in longitudinal panel cracking and lane-shoulder drop-off. The presence of localized depressions in the shoulder adjacent to transverse pavement joints (see Figure 7.6) may indicate the presence of voids under the slab corners and the potential for corner cracking.

Figure 7.6 Pumped foundation material along longitudinal joint at MnROAD Cell 40

Minnesota DOT

Another possible source of poor slab support at the corners is slab curl/warp, particularly when the slab rests on (but is not bonded to) a stabilized base. When the slab surface is cooler and/or drier than the slab base, upward curling and warping may develop, resulting in loss of slab corner support, as shown in Figure 7.7. Some built-in temperature and moisture gradients may be present if the concrete hardened in a flat condition in the presence of temperature and moisture gradients.

Figure 7.7 Schematic showing upward curl and loss of support near slab corners in response to moisture gradient (drier surface) and/or temperature gradient (cooler surface)



Cause

Factors that can contribute to corner cracking due to corner loading with loss of panel support are discussed below.

- **Heavy Vehicle Loads**: Corner cracking is caused by the repeated application of vehicle loads on a poorly supported corner of the slab.
- **Poor Load Transfer – Transverse Joint**: Differential deflections of the panels on either side of the transverse joint initiate the pumping mechanism that results in erosion of the foundation and loss of slab support. Differential deflection is caused by the passage of heavy wheel loads over transverse joints with poor load transfer mechanisms (i.e., insufficient aggregate interlock and some joint opening, or an inadequate or failing dowel load transfer system).
- **Poor Panel Edge Support (Longitudinal Joint)**: Differential deflection across the longitudinal joint (usually the lane-shoulder joint) can initiate a pumping mechanism that results in erosion of the foundation and loss of slab support. Differential deflection along the longitudinal joint is caused by the passage of heavy wheel loads along a longitudinal joint between slabs or materials that provide little vertical restraint to each other (e.g., untied concrete lanes or asphalt shoulders with a butt joint).
- **Free water at Slab/Subbase and its Subbase/Subgrade Interface**: Free water in the pavement system can be moved with great speed and pressure under the action of heavy wheel loads near transverse and longitudinal joints with poor load transfer characteristics. This rapid, high-pressure movement of water can erode foundation materials, especially those that are unbound.
- **Use of Erodible Foundation Materials**: Weakly bound and unbound foundation materials are especially susceptible to erosion by pumping of water in the pavement structure by heavy vehicle loads at joints with poor load transfer. Their erosion leads to loss of pavement support and eventual corner cracking.
- **Slab Curl and Warp**: Temperature and moisture gradients through the thickness of the concrete slab produce slab deformations called curling and warping (see Chapter 10). When the slab is supported by (and not bonded to) a relatively stiff material (e.g., new construction on an asphalt- or cement-treated base, unbonded concrete overlays of any pavement type, etc.), portions of the panels near the panel edges may become unsupported during periods of dry, cool weather when the slab curls/warps upward at the corners. Heavy vehicle loads and the self-weight of the unsupported slab corners can produce slab stresses that result in corner cracking.

- **Inadequate Compaction**: Loss of slab support may be due to post-construction settlement of the base, subbase, or roadbed soil after inadequate or variable compaction during construction. Other possible contributing factors include variability in roadbed soil materials and segregation of granular base materials. These factors can lead to locally increased slab deflections and slab stresses under traffic loads and can result in transverse, longitudinal, or diagonal cracking. It is possible that they could also result in corner cracking.
- **Subgrade Volume Stability**. Some foundation materials (e.g., swelling/collapsing soils and frost-susceptible soils) are susceptible to volumetric changes. Swelling soils swell and shrink due to changes in the moisture content and are more commonly associated with heavy clay materials with high plasticity indices and liquid limits (e.g., AASHTO A-6 and A-7 classifications). Frost heave refers to the distortion or expansion of the subgrade as a result of the growth of ice lenses under freezing temperatures and requires the contribution of three factors: 1) a sufficiently cold climate to allow freezing temperatures to penetrate the subgrade, 2) a supply of moisture into the freezing zone, and 3) a susceptible soil (typically low plasticity silts, which exhibit a combination of capillarity and permeability that feeds the growth of the ice lenses) (ACPA 2007a). When these types materials are uniformly distributed along a project, the entire pavement structure generally moves up or down uniformly. However, when these materials are present in local deposits, differential movements take place along the length of the pavement and may result in transverse, diagonal, or longitudinal slab cracking. It is possible that they could also result in corner cracking. Additional details on frost heave and swelling soils are found in Chapter 12.

Prevention

Table 7.3 provides a summary of general recommendations for preventing loss of slab support that can lead to the development of longitudinal slab cracking.

Note that there are competing interests in selecting a concrete pavement subbase type. For example, stabilized subbases are more resistant to erosion and loss of support than granular materials but stabilized bases are also generally more stiff and can result in areas of poor slab support due to curling and warping. A compromise or balance must be struck between these considerations. Fortunately, a wide range of base materials have been used successfully beneath concrete pavements (in combination with other design features) to provide erosion-resistant and uniform support, a good construction platform, and acceptable levels of curl/warp and frictional stresses.

Table 7.3 Considerations for addressing corner cracking caused by vehicle loads and poor slab support

Pavement Layer	Aspect	Issue	Considerations
Surface Layers (Concrete Pavement and Adjacent Lanes or Shoulder)	Transverse joint load transfer	Relative displacement across the joint initiates pumping mechanism	Use properly designed and constructed load transfer systems (e.g., dowel bars) in all transverse joints
Surface Layers (Concrete Pavement and Adjacent Lanes or Shoulder)	Longitudinal joint edge support	Relative displacement across the joint initiates pumping mechanism	Provide good edge support along all longitudinal joints Examples include properly tied longitudinal joints between concrete lanes or concrete lanes and concrete shoulders, using widened concrete lanes adjacent to the shoulder, etc. Avoid the use of asphalt or granular shoulders adjacent to normal-width paving lanes in the presence of heavy truck traffic and erodible foundation materials
Surface Layers (Concrete Pavement and Adjacent Lanes or Shoulder)	Joint Sealing	Entry of water through unsealed joints (especially longitudinal joints) can result in pumping	Seal all pavement joints (especially longitudinal lane-shoulder joints) in wet climates, especially when heavy trucks and erodible foundation materials are present
Surface Layers (Concrete Pavement and Adjacent Lanes or Shoulder)	Concrete Coefficient of Thermal Expansion (CTE)	High CTE and contraction increases slab curl for any temperature gradient, especially for longer panels	Use concrete with the lowest possible CTE (i.e., concrete aggregate with low CTE, such as limestone, basalt, and granite rather than sandstone and quartz) Avoid using concrete with high CTE when using panels with dimensions greater than 12 ft, especially on very stiff or stabilized base layers

Pavement Layer	Aspect	Issue	Considerations
Surface Layers (Concrete Pavement and Adjacent Lanes or Shoulder)	Concrete Shrinkage	High shrinkage characteristics increase slab warping deformations for any moisture gradient	Use concrete mixtures with low shrinkage potential (i.e., the lowest feasible content of cementitious material and the lowest feasible water content)
Base or Subgrade	Erodibility	Pumping and erosion of base or subgrade leads to unsupported conditions	Use bound (stabilized or treated) base and subgrade materials, especially for facilities that carry heavy truck traffic in wet climates If unbound aggregate base layers are used, limit fines (material passing the No. 200 [0.075mm] sieve) to 10% or less (but consider minimum fines required to achieve stability during construction operations)
Base or Subgrade	Compaction	Poor or inadequate compaction can lead to post-construction settlement of the base, reducing the support provided to the slab	Ensure that all unbound foundation layers are compacted to the specified target densities Ensure base course is homogenous and not segregated when placed
Subgrade*	Swelling Soils*	Subgrade volumetric changes due to variations in subgrade moisture contents	Remove and replace small areas of swelling soils Compact at 1–3% above optimum moisture content (AASHTO T 99) Consider use of soils stabilization and membranes

Pavement Layer	Aspect	Issue	Considerations
Subgrade*	Frost Heave*	Subgrade volumetric changes due to frost penetration and growing ice lenses in subgrade	Compact slightly wet of optimum moisture content (AASHTO T 99) Use non-frost-susceptible materials within the depth of frost penetration Protect (cover) frost-susceptible soil with sufficient thickness of non-frost-susceptible material
Pavement Structure (General)	Joint Sealing and Drainage	Water that enters and remains in the pavement structure can result in pumping	Seal concrete pavement joints (especially the lane-shoulder joint) and/or provide edge drains or day lighted aggregate base to quickly remove water from the pavement structure

*More detailed information provided in Chapter 12

Other Corner Cracking Mechanisms

A less common mechanism of corner cracking is localized restraint of transverse joint closure due to infiltration of mortar or other incompressible materials into the transverse joint at the longitudinal joint during placement of an adjacent lane or shoulder. Infiltration may occur when several weeks or months pass between construction stages, and the joints in the first stage placement are opened (due to shrinkage or cool weather) when the adjacent lane or shoulder is placed. Cracking typically occurs during the next period of very warm weather when the panels in the first stage placement expand and the closure of the joints is restrained by the infiltrated material. If the amount of infiltrated material is small, a joint spall may result; if the amount and depth of infiltrated material is great, a small corner crack may develop (see Figure 7.8).

Figure 7.8 Small corner crack, possibly due to infiltration of mortar during adjacent lane placement

National Concrete Pavement Technology Center

A similar mechanism can occur without paste or mortar infiltration when adjacent lanes are placed during significantly different environmental conditions. If the first stage placement joints have activated and opened when the second stage is placed and tied to the first stage (e.g., first stage placement during warm weather and second stage placement during cool weather), the second placement will restrain subsequent slab expansion and joint closure in the first placement and can result in a corner crack or a longitudinal crack (as described in Chapter 6).

Another mechanism that can produce uncontrolled cracking that resembles corner cracking involves joint sawing operations. When transverse joints are sawed late in the sawing window, the joint will sometimes propagate randomly ahead of the saw blade as the saw approaches the panel edge. This may not be a corner crack at first if the sawed joint doesn't fully activate but it is likely that the joint will eventually activate under the saw crack (especially with traffic loadings), resulting in a corner crack.

Table 7.4 summarizes the key points related to the causes and prevention of corner cracking in concrete pavements. This is a broad summary, with more detailed information provided in the previous discussions.

Table 7.4 Overall summary of causes and prevention or mitigation of corner cracking

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Corner Cracking Due to Heavy Loads and Unsupported Slab Corner	Repeated heavy load applications Inadequate load transfer and edge support Free water in foundation materials Pumping and erosion of foundation materials Slab curl/warp Inadequate compaction of foundation materials Volumetric changes in foundation materials (swelling, frost heave, etc.)	Incorporate mechanical load transfer devices (e.g., dowel bars) in transverse joints Provide good edge support through tie bars and/or widened lanes Provide pavement drainage (edge drains or day lighted permeable base) to reduce moisture in foundation materials Design, construct, and maintain sealed joint systems (reservoir shape factor and sealant selection) Alternatively, use narrow, unsealed single-cut joints with effective drainage to remove infiltrated water, where appropriate. Design total pavement structure to exceed frost depth	Use concrete with low CTE and low shrinkage potential using reduced paste Use erosion-resistant materials in foundation layers Consider stabilized or treated base materials Consider stabilization of plastic subgrade soils Limit fines in unbound base layers Ensure foundation layers are homogeneous Remove and replace small areas of swelling soils	Compact all foundation layers to target densities Compact swelling soils and frost-susceptible soils slightly wet of optimum moisture content Properly seal all surface joints and maintain joint seals Maintain pavement drainage systems

Distress in Concrete Pavement	Contributing Causes	Prevention: Design	Prevention: Materials	Prevention: Construction
Corner Cracking Due to Construction Staging, Infiltration of Mortar in Open Joints	New lanes or shoulder placed adjacent to existing pavement with open joints Infiltration of mortar from new concrete in existing pavement joints, restraining them from closure during warm weather Tied lanes or shoulders	N/A	N/A	Construct adjacent lanes when existing lane joints are all tight or are (slightly) uniformly open Caulk or seal vertical faces of transverse joints in prior concrete placements before placing adjacent concrete lanes or shoulders

6. Treatment and Repairs

General repair methods and maintenance approaches to prevent and repair corner cracking are described in the following sections.

Full-Depth Repair (FDR)

The only proven and generally accepted repair for corner cracking is FDR. FDR and slab replacement are commonly used to address deteriorated longitudinal cracks in concrete pavements. These repairs consist of the removal of isolated deteriorated areas through the entire thickness of the existing slab and replacement with a concrete replacement material.

Many agencies perform FDRs on one or both sides of the affected joint (as needed) and over the full width of the paving lane; however, some agencies have successfully used half-lane-width repairs because corner cracking extends across one-half of the lane width or less (by definition). In either case, the transverse joint should be re-established with mechanical load transfer devices and the remaining repair boundaries are typically tied or doweled, as appropriate. Concurrent repair activities may include foundation repairs, stabilization (grout or urethane injection) of surrounding slab areas, dowel bar retrofit of other joints, provision of edge support, and diamond grinding.

The *Concrete Pavement Preservation Guide* (Smith et al. 2014) provides details on the design and construction of effective FDRs and many of the concurrent repair techniques described above.

Pavement Preservation

The development of corner cracking can be delayed or prevented using a combination of conventional pavement preservation techniques, including slab stabilization, dowel bar retrofit, cross-stitching of longitudinal joints, addition of edge beams or tied concrete shoulders, installation of edge drains, and installation/replacement of joint sealant.

Conditions that may warrant pavement preservation to prevent the development of corner cracking include:

- Evidence of pumping near pavement joints (i.e., deposit of foundation materials on the pavement shoulders near transverse joints),
- Development of transverse joint faulting, or
- Observation of high relative deflections across transverse joints during FWD testing (i.e., poor load transfer characteristics).

The source(s) or cause(s) of the observed condition must be determined and addressed through the proper combination of preservation activities, which must be chosen to repair damage that has already taken place (e.g., foundation erosion and loss of pavement support) as well as to prevent future damage from taking place.

Appropriate treatments may include the following.

- Slab stabilization – injection of cement grout or urethane to fill voids that have developed under slab corners
- Load transfer restoration – installation of dowel and tie bars to minimize the differential deflection that takes place when loads cross transverse joints and travel along longitudinal joints
- Addition of edge beams or tied concrete shoulders to provide pavement edge support

- Installation of edge drains – trenching along the pavement edge and installing drain systems to intercept, collect, and remove infiltrated water
- Joint (re-)sealing/(re-)filling – cleaning and refacing existing joint cuts (as necessary), and filling/sealing them with appropriate sealant materials

Details concerning the proper techniques for performing most of these activities (all except the addition of edge beams and tied concrete shoulders) are available in the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

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CHAPTER 8. SPALLING—TRANSVERSE AND LONGITUDINAL JOINTS AND CRACKS

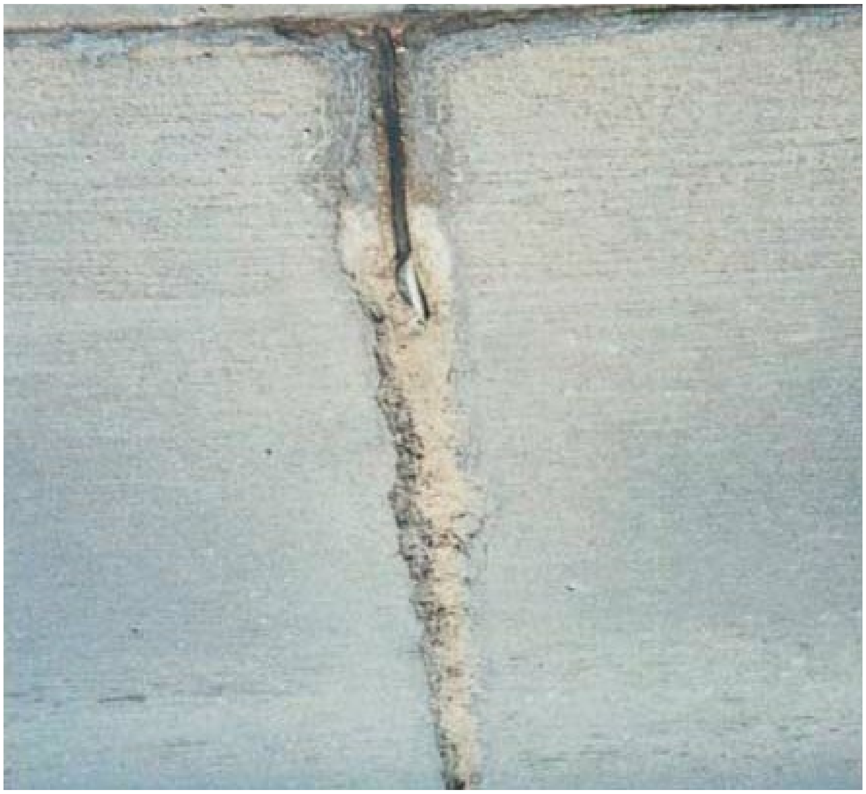
1. Description

Transverse and longitudinal joint/crack spalling in concrete pavements is one of the most common, if not the most predominant, concrete pavement distresses. Joint and crack spalling is deterioration of the opening, which refers to cracking, chipping, or fraying of the concrete slab joint or crack edges of the transverse and longitudinal joints and cracks. The spalling may develop predominantly in the top few inches of the slab, or may develop at a greater depth below the surface, depending on the environmental conditions, eventually reaching full pavement depth. Spalling problems include loose debris on the pavement, shallow vertical drops, and roughness (See Figure 8.1). Spalling can be expanded both in width and depth through continuous deterioration.

Figure 8.1 Common spalling of joints and cracks

a. Freeze-thaw damage

Kevin McMullen, Wisconsin Concrete Pavement Association



b. Saturated joint backer rod damage

Peter Taylor, National Concrete Pavement Technology Center

c. Saturated joint with unsound aggregate

Snyder & Associates, Inc.

d. Incompressible joint damage
Caltrans

e. Deflection spalling from heavy vertical loads
Dale Harrington, HCE Services

f. Early saw joint raveling
National Concrete Pavement Technology Center

g. Chloride penetration
John Donahue, Missouri DOT

h. Misaligned dowel bar
Jeff Uhlmeyer, Washington State DOT

2. Severity

The following information in Table 8.1 was obtained from the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014).

Table 8.1 Severity levels and measurements

Distress	Description	Measurements
Longitudinal Joints	Cracking, breaking, chipping or fraying of slab edge within 1 foot (0.3m) from the face of the longitudinal joint	**Low:** Spalls less than 3 inches wide measured to the face of the joint with loss of material and no patching, or spalls with no loss of material and no patching. **Moderate:** Spalls 3 to 6 inches wide measured to the face of the joint with loss of material. **High:** Spalls greater than 6 inches wide measured to the face of the joint with loss of materials, or spalls broken into two or more pieces, or spalls containing patch materials. Only record spalls that have a length of 4 inches or more. When a crack is within 1 foot of a joint for only a portion of its length, it should be recorded as a spall.

Distress	Description	Measurements
Transverse Joints	Cracking, breaking, chipping, or fraying of slab edge within 1 ft (0.3 m) from the face of the transverse joint	**Low:** Spalls less than 3 in. wide measured to the face of the joint with loss of material and no patching, or spalls with no loss of material and no patching. **Moderate:** Spalls 3 to 6 in. wide measured to the face of the joint with loss of material. **High:** Spalls greater than 6 in. wide measured to the face of the joint with loss of materials, or spalls broken into two or more pieces, or spalls containing patch materials. A joint is affected only if the total length of spalling is 10% or more of the length of the joint. Rate the entire transverse joint at the highest severity level present for at least 10% of the total length of the spalling. Record length in meters of the spalled portion of the joint at the assigned severity level for the joint. When a crack is within 1 foot of a joint for only a portion of its length, it should be recorded as a spall only for that portion.

Source: Modified from Miller and Bellinger 2014

3. Testing

Following the visual on-site examination of the severity of the spalling, it may be necessary to conduct certain evaluation tests to try to isolate the various factors that may have caused the spalling distresses. More detailed information regarding testing is included in Chapter 19.

Field Tests

Coring

It is advisable to take core samples in the spall to determine the depth of degradation and the type and condition of the concrete and subbase below the spalled area. This could determine the type and extent of needed repairs.

Magnetic Imaging Tomography

This test may be necessary to evaluate dowel bar placements and misalignments, if that is suspected.

Falling Weight Deflectometer (FWD)

A FWD test may be necessary for assessing uniformity and structural adequacy of the subbase or subgrade when vertical pavement displacement is prevalent in the existing slab.

Laboratory Tests

Various factors affect the necessity for additional laboratory pavement evaluation tests. If the pavement is less than 10 years old and has marked spalling, it would be advisable to conduct additional laboratory tests to determine if it is cost-effective to complete specific repairs. Such tests would consist of petrographic analysis to determine the adequacy of the air entrainment system, aggregate coefficient of thermal expansion (CTE), and soundness or durability of the aggregate.

4. Identification of Causes

Table 8.2 provides descriptions of the physical and material/chemical factors that cause spalling. A more detailed description of causes is provided under the evaluation section below.

Table 8.2 Summary of physical and material/chemical causes of spalling

Distress	Category	Description
Spalling (Physical)	Inadequate Consolidation	Inadequate portland cement concrete (PCC) consolidation during construction, causing weak concrete
Spalling (Physical)	Infiltration	Infiltration of incompressibles into poorly sealed or unsealed joints
Spalling (Physical)	Compression Shear	Compression shear from deflection of the slab, lack of load transfer, or lack of subgrade support
Spalling (Physical)	Chipping/Fraying	Early sawing of the joint which chips or frays the edges of the joint
Spalling (Physical)	Moving Dowels	Dowel bar movement from misaligned dowels
Spalling (Physical)	Curling and warping movement	Occasionally spalling can occur from curling and warping movements but it is not typical. High surface temperature expansion (curling) in combination with high bottom moisture expansion (warping), as a result of a wet subbase or subgrade, causes compression in the top of the joint or crack. Details of curling and warping are covered in Chapter 12.
Spalling (Material or Chemical)	Freeze-Thaw Damage	Freeze-thaw damage to the paste of the concrete, resulting from either or both of the following: • Poor air entrainment system not providing adequate storage for frozen moisture during freezing cycles • Saturated concrete joints/cracks from poor drainage system and the chemical breakdown of the concrete from deicing salts such as calcium and magnesium chloride
Spalling (Material or Chemical)	Poor Durability	Poor soundness and durability of the aggregate beyond D-cracking and alkali-silica reaction (ASR) is the breakdown and disintegration of aggregates from weathering (wetting/drying and freezing/thawing) can cause pavement distress at the joint Durability and soundness are terms typically given to an aggregate's weathering resistance characteristic

Distress	Category	Description
Spalling (Material or Chemical)	Thermal Expansion	High CTE of the aggregate results in higher compressive stresses at the joint or crack

5. Evaluation

The following are detailed descriptions of the various causes (or distress mechanisms) of spalling and the prevention, or reduction of, spalling.

Freeze-Thaw Damage

Cause

Joint deterioration (spalling) from freeze-thaw damage in cold weather states is attributed to saturated joints with moderate to low air entrainment content (less than 5%) and being exposed to high concentrations of deicing chemicals, as described in the *Interim Guide for Optimum Joint Performance of Concrete Pavements* (Taylor et al. 2012), *Constructing Concrete Pavements with Durable Joints* (Taylor 2014)*, and Guide to the Prevention and Restoration of Early Joint Deterioration in Concrete Pavements* (Weiss et al. 2016). Typically, damage is in the paste, resulting in thin flakes of mortar that form parallel to the exposed joint face. Figure 8.2 shows freeze-thaw damage with the flaking of the paste and the loss of the damaged paste that leaves the remaining aggregate.

Figure 8.2 Freeze-thaw damage

a. Damage of the hardened paste

Peter Taylor, National Concrete Pavement Technology Center

b. Loss of the damaged paste over time, leaving exposed aggregate
National Concrete Pavement Technology Center

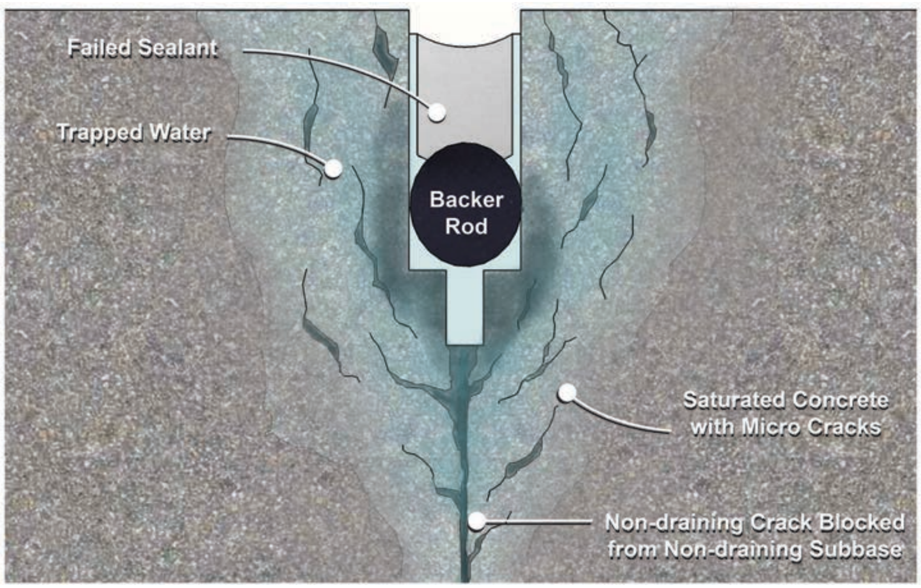
Causes

Saturation. Saturated frost damage is due to expansion (9 percent) of water in saturated capillaries of the concrete as it freezes. As the water in the concrete freezes, it produces pressure in the capillaries and pores of the concrete. If this pressure exceeds the tensile strength of the concrete, the concrete will crack. Cycles of freezing and thawing open these cracks more and allow water to penetrate into the concrete, which results in the concrete deteriorating incrementally. Concrete that is highly saturated is prone to accelerated damage. Common characteristics or practices on pavement leading to saturation include the following:

- Pavement saturated for long periods regardless of the source of water.
- Poor drainage system that does not allow for the drainage of the subbase system, through either daylighting the subbase or improper maintenance of the subdrained structure. This results in transverse and longitudinal joint deterioration from freeze thaw damage.
- High permeability of the concrete mixture which increases the rate of saturation.
- The use of significant quantities of aggressive deicing salts.
- Highly absorbent and unsound aggregate that is saturated and has low freeze thaw resistance.

As shown in Figure 8.3, water can be present in a pavement system because of inadequate surface or subsurface drainage, or high water table, or no crack to drain the water, which results in water being trapped in the bottom of the saw cut and under the backer rod.

Figure 8.3 Image of micro cracking in concrete pavement joint from saturation in freeze-thaw climates due to failed sealant, non-draining base, and placement of a backer rod



a. Poor joint sealant leading to saturation in concrete joint, resulting in micro cracking
Snyder & Associates, Inc. after Jason Weiss

b. Longitudinal freeze damage from backer rod trapping water beneath sealant
National Concrete Pavement Technology Center

Increasing the saturation of concrete will also decrease its ability to resist freezing because there is more water in the system than can be accommodated when freezing occurs (PCA 2002). Concrete that is less than 85%

saturated can survive, while saturation greater than this will likely result in permanent damage, regardless of the entrained air (Taylor 2014).

Chemical deicing: Deicing salts can aggravate freeze-thaw deterioration by attracting and absorbing water, and thus increasing saturation. Calcium and magnesium chloride salts, and to a lesser degree sodium chloride, keeps the pavement wetter, particularly at the joints, until it's washed off in some future event. The additional problems caused by deicers includes the buildup of osmotic and hydraulic pressure in excess of normal hydraulic pressures produced when water in concrete freezes. This can come from the expansion of crystallized salt as water evaporates in the absence of freezing.

Sodium chloride or rock salt has little chemical effect on concrete. However, calcium chloride, and most notably magnesium chloride, not only retain the water but can chemically attack the concrete. Calcium and magnesium chloride result in the formation of calcium oxychloride, which is expansive and can cause damage and cracking of the concrete. This occurs when the calcium hydroxide (CH) present in a typical hydrant cement paste reacts with calcium chlorides ($CaCl_2$) to form calcium oxychloride. The phase change to calcium oxychloride is highly expansive, with the resulting damage to the hydrated cement paste likely due to crystallization pressures.

Deterioration is sometimes first observed as shadowing (see Figure 8.4) or a darkening of a zone a few inches on either side of a joint.

Figure 8.4 Evolution of joint deterioration from shadowing (left) to high severity (right)

National Concrete Pavement Technology Center

Marginal Soundness of Aggregate: Aggregate quality issues beyond typical D-cracking and ASR aggregates (see Chapter 4 for details of D-cracking and ASR aggregates) have resulted in spalling in concrete pavement joints when subject to deicing under wet conditions for a number of years. Figure 8.5 shows a concrete pavement, in a cold weather state, with both transverse and longitudinal spalling due to repeated freeze-thaw cycles of wet joints over a period of 15 to 20 years.

Figure 8.5 Saturated transverse and longitudinal joints with unsound aggregate

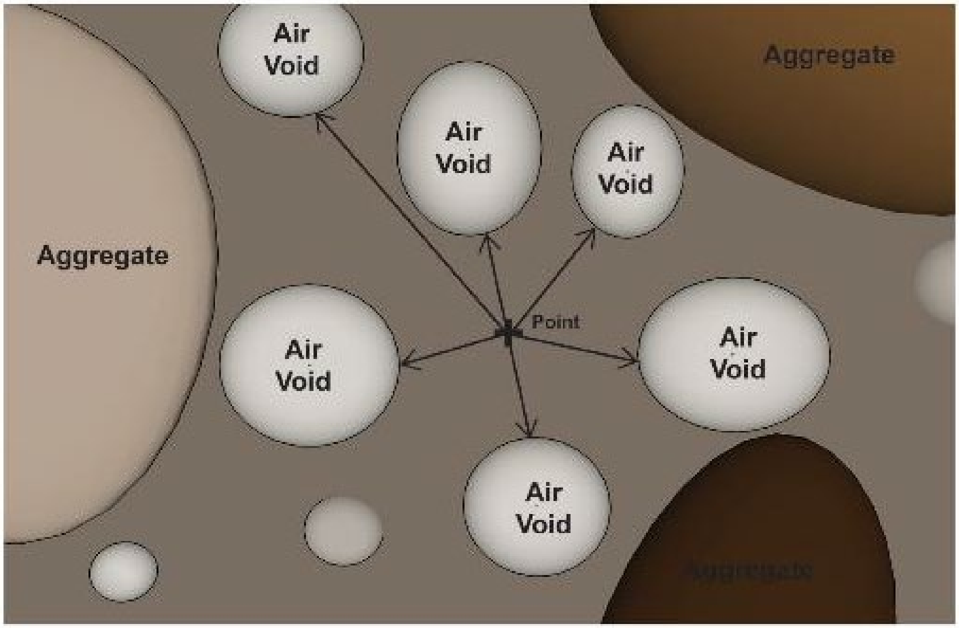
Dale Harrington, HCE Services

D-cracking patterns did not exist in the pavement but tests on the limestone course aggregate showed it had medium absorption rates and medium-size pores, which resulted in the breakdown of the aggregate before the design life was reached. This aggregate did not meet the sounding test, freezing and thawing resistance, or degradation resistance test that are now commonplace.

Poor Air Void System: Air voids in concrete play a critical role in improving concrete pavements' durability by reducing their susceptibility to freeze-thaw damage. When most of the cement in a concrete mixture has reacted with water in the mixture, some of the remaining water is lost to evaporation, leaving behind capillary pores. Any remaining water can move through these pores. If the temperature drops below freezing, water in the capillary pores turns to ice and expands by 9%. As the expanding water forces its way through the pores, it exerts pressure on the surrounding hardening cement paste. Entrained air voids act as pressure relief valves, providing space for the water/ice to expand and relieve pressure on the surrounding concrete. Without adequate air voids, the pressure will cause hardened cement paste to crack initiating early pavement deterioration.

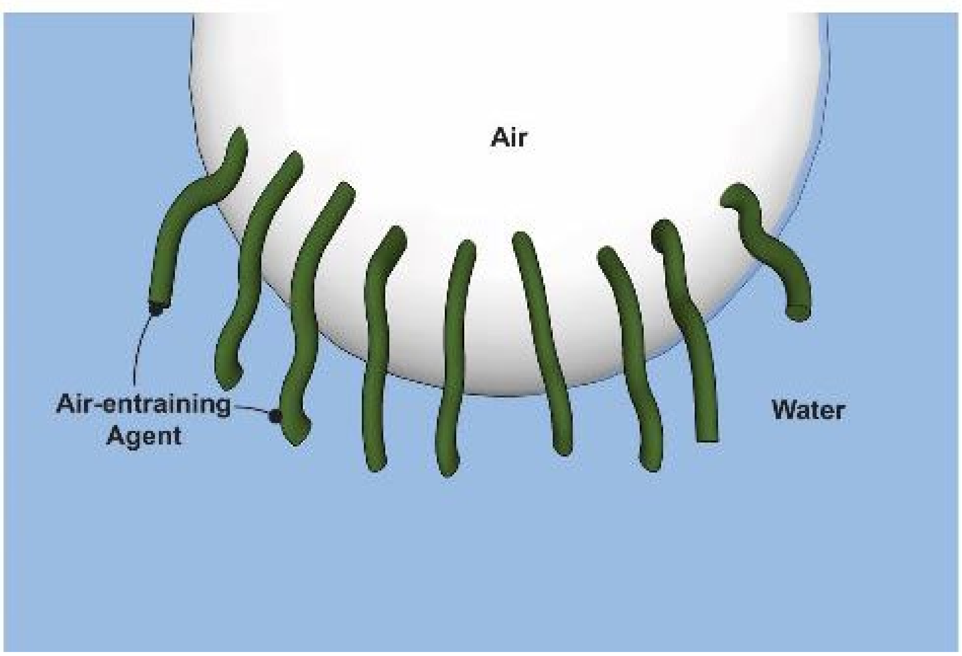
Air voids are created through the proper mixing of the concrete. Uniformly distributed air voids consist of many small, closely spaced voids that provide the greatest protection against freeze-thaw damage. Small, tightly spaced air voids are better for storing water than larger air voids. When this does not occur, the total air volume measurement could be low or when the air volume measurements show an adequate total air volume, it could be the result of unwanted entrapped air. Stabilizing air bubbles in the concrete is accomplished through air-entraining agents such as pine wood resins (vinsol resins) and other synthetic detergents. This stabilizing of the millions of tiny air bubbles is accomplished through soap-like coatings around the air bubbles. Molecules of air entraining agents are attracted to water at one end and to the air at the other, reducing surface tension at the air-water interface. The ends that protrude into the water are attracted to the cement particles and the particles adhere to the entrained air (see Figures 8.6 and 8.7).

Figure 8.6 Spacing factor is the average distance from any point to the nearest air void



Celik Ozyildirim

Figure 8.7 Stabilization of air voids by air-entraining admixture molecules



After Thomas and Wilson 2002, Portland Cement Association, Illustrator/previous publisher of this rendition unknown

For concrete pavement that is exposed to deicing chemicals or high water saturation, the Concrete Pavement Technology Center recommends a minimum of 5% to 8% air content in the hardened concrete to prevent freeze-thaw damage. In addition, a spacing factor equal to or below 0.008 inches is recommended. Poor entrained air is considered when the spacing factor is greater than 0.008 inches to 0.04 inches. See the

Integrated Materials and Construction Practices for Concrete Pavements manual (Taylor et al. 2007) for more detailed information.

Loss of proper air entrainment can be attributed to the following:

- Improper mixing time of the concrete; typically shorter mixing times can increase the problem.
- Some air entrainments may not develop stable air bubbles in short mixing windows. These materials may be satisfactory in ready mix applications but not in central mixers. Additional mixing time may be required or a change of air-entraining material may be mandated.
- High vibrations during concrete pavement placement. Over-vibrations can cause the loss of air voids, as much as 3% of loss has been recorded between the front and back of the paver.
- Improper entrainment agents that do not stabilize the bubbles correctly, or accumulations of air voids around aggregate particles, known as clustering. Research has indicated that this is most likely to occur with the use of non-vinsol air-entraining admixtures and the addition of water to the mixture after initial mixing (Taylor and Kosmatka 2007). Extended mixing will exacerbate the problem.
- Secondary ettringite deposits fill the air void system in the lower pavement section as a result of saturated subbase conditions.
- The use of fly ash with high carbon content and silica fume in the mixture. (Some loss of air content can occur from slag with increasing fineness.)
- Transportation with long hauls, pumping of concrete, and excessive finishing.

Prevention

There are four primary strategies for preventing or reducing freeze-thaw damage to concrete joints: preventing saturation, ensuring adequate air entrainment, reducing concrete permeability, and increasing supplementary cementitious material (SCM) use.

Prevent saturation: Attention to detail regarding how water will be prevented from collecting and staying in a joint is critical. This includes if and how the joint is sealed and whether the water can penetrate into the joint and remain. If the base or support layer is drainable, the crack below the saw joint will not retain water and thus prevent saturation of the joint. In addition, adequate drainage of the subbase layer prevents secondary ettringite deposits from filling air voids in the bottom portion of the concrete pavement. Many concrete pavements' granular subbase systems are drained by subdrain tile located in the shoulder of the pavement.

To prevent the issue of subdrain outlets plugging and not being checked periodically, the granular drainage subbase system should be extended out beyond the shoulder intersecting with the ditch.

Entrapped water in transverse joints can be a predominant saturation factor but also occurs in longitudinal joints. The elevation of some longitudinal joints is, at times, lower than transverse joints and due to the pavement cross slope, water from the transverse joint can accumulate in the longitudinal joint. When water cannot drain into an adequate subbase layer that has a proper outlet, spalling issues may also result in the longitudinal joint. This is why it is important to have an adequate drainage layer underneath the pavement to prevent water trapping in the joint.

Ensure adequate air entrainment: The resistance of concrete to freezing and thawing in a moist condition is significantly improved by the use of intentionally entrained air. Entrained air voids act as empty chambers in the paste for the freezing and migrating water to enter, thus relieving the pressure in the capillaries and pores and preventing damage to the concrete. To help ensure a proper air entrainment system, the following should be achieved:

- Proper mixing and time of mixing of the concrete.
- Adequate air entrainment admixtures to stabilize air bubbles.
- Proper control of the machine vibration during placement.
- Check the volume of the air voids in front of the paver at regular specified intervals and at least periodically during the pour behind the paver. The key is to test a representative batch of concrete in front of the paver and then test that same batch behind the paver. Testing the same truck is important. This testing does not need to be done on every batch or air test. It should be done two or three times on the first day of production and then a lesser amount when there is confidence in the consistency. The air testing in front of paver can be relied on for the quality measure once a stable air system has been established through the paver. Also, an air loss correction factor should be established between the front and back of the paver.
- Lowering the cement content helps maintain air void systems.
- Adequate drainage of the subbase helps prevent the formation of secondary ettringite crystals that can form in the air voids in the bottom portion of the concrete pavement, particularly in wet conditions. There is debate if packing the air voids with ettringite is sufficient enough to render the voids useless over time. It is recommended to view the formation of secondary ettringite as a symptom of a continuous wet subbase or subgrade, which should be drained. The amount of filling of the ettringite crystals varies; however, if the air void system was originally marginal, there is probable cause for concern If not drained, the packing of the air voids with ettringite will likely continue, placing the air void system in jeopardy. (See Figure 8.8.)

Figure 8.8 Secondary ettringite crystals in air voids

Gerald Moulzhoff

Water reducers and retarders acting as hydration stabilizers, along with longer mix times, can improve air content.

Until recently, the air meter utilized for testing the air content in concrete measured only the percent of total air in the concrete and not the spacing factor. The Super Air Meter (SAM) (Figure 8.9), which was developed by Tyler Ley at Oklahoma State University, is an FHWA-tested device that measures both the air void spacing and air volume of plastic (fresh) concrete in about 10 minutes.

Figure 8.9 Super Air Meter (SAM)

Tyler Ley

By measuring the actual air void spacing in the fresh concrete, the meter helps users better understand the freeze-thaw durability of concrete before it is placed. Numerous state DOTs throughout the US are beginning to use the SAM and are formulating their measurement procedures.

Reduced concrete permeability: Concrete pavement with low permeability is better able to resist the penetration of water and, as a result, it performs better when exposed to freeze-thaw cycles. The permeability of concrete is directly related to its water to cementitious material (w/cm) ratio and the paste content; the lower the w/cm and paste content, the lower the permeability of concrete. Reducing permeability can be achieved by the following:

Limiting the maximum w/cm ratio consistently to below 0.45. Ideally, the w/cm ratio should be closer to 0.42. Lower the w/cm ratio by removing water and not adding more cementitious material. In part, lowering the cement content can be accomplished by optimizing the aggregate gradation. For a given w/cm, reducing the cementitious material content through increasing aggregate volume will reduce the ultimate shrinkage of the concrete. It also helps to provide internal resistance to shrinkage. However, there are practical limitations in reducing the w/cm ratio. If the w/cm drops much below 0.40, autogenous shrinkage, due to chemical shrinkage and self-drying of the paste as water is consumed in hydration, becomes prominent. According to the PCA's *Design and Control of Concrete Mixtures* (Kosmatka and Wilson 2016) autogenous shrinkage is most predominant in concrete with a w/cm ratio under 0.42. Early age shrinkage, such as autogenous shrinkage, is important to control because it occurs at a time when concrete is developing stiffness at a faster rate than strength. As such, the development of early age cracking can occur when not controlled. Caution: Low w/cm (0.40 and below) may have no bleed water and therefore requires an immediate application of a low moisture loss curing membrane, such as water curing. Low w/cm can be fragile mixes that require special attention during handling and placement. The working window tends to be more restrictive, and can be difficult to place and finish. It is critical to maintain the aggregate moisture at saturated surface dry (SSD) or greater to prevent moisture loss through absorption.

Implementing rigorous curing techniques inside joint curing and on the joint face is currently in an experimental phase to determine if it is effective, practical, and cost-effective.

Use of surface coatings, such as penetrating sealers, is being experimented with in Iowa. There is early evidence that penetrating sealers could help reduce the rate of ingress of water into the concrete at the joint. According to the National Concrete Pavement Technology Center, siloxane-base materials have the history of reducing joint permeability but if used, they should be replaced periodically approximately every 5 to 7 years.

Use of well-graded aggregate and lower paste contents (aggregate is less permeable than cement paste).

When properly applied, a high-quality curing compound slows the loss of moisture from the pavement to the atmosphere and significantly lowers permeability in the upper wearing surface of the slab.

Increased SCM Use: The use of SCMs help reduce permeability in the concrete and also has the ability to reduce unwanted chemical reactions such as the production of oxychlorides.

When possible, use appropriate SCMs such as fly ash and/or ground granulated blast furnace slag at appropriate dosages. SCMs convert calcium hydroxide (CH) in the cement to calcium silicate hydrate (CSH), which not only improves strength but lowers the permeability of concrete.

By tying up or converting some of the CH with the use of fly ash or slag, the formation of damaging calcium oxychloride is substantially reduced by as much as 45%.

Joint Compression – Incompressible Materials

Transverse and longitudinal joints are designed and constructed in JPCP to allow slab expansion and contraction, and to prevent cracking. However, incompressible materials can lead to compression stresses at the face of the joint which can cause spalling.

One purpose of sealing joints and cracks in concrete pavement is to reduce infiltration of moisture and incompressible materials for improved pavement performance. Incompressible material fills the joint or crack openings and create excessive stresses that may cause spalling. (See Figures 8.10 and 8.11.)

Figure 8.10 Incompressible damage to concrete joint

National Concrete Pavement Technology Center

Figure 8.11 Compression spalling

http://wikipave.org/index.php?title=Joint_Sealant_Evaluation, American Concrete Pavement Association

Cause

Incompressible materials such as small aggregates can enter into unsealed joints causing excessive stresses along the joint face of the concrete when the joint contracts (Figure 8.11). The result can be fracturing of the joint near the top of the slab, particularly with transverse joints. Joints become wider over time (particularly the first 5 years) as the panels shrink. Intrusion of aggregates into the joint has also occurred in longitudinal joints near the outside edge of the slab with granular shoulders. As concrete pavement ages, the slab is subjected to continued shrinkage. This horizontal movement from contraction and expansion with temperature and moisture fluctuations, along with the vertical impact of repeated traffic loads, can cause excess compressive stress. The result can be micro cracking, which eventually turns into spalling.

Prevention

There are two ways to prevent incompressibles from entering an unsealed joint: joint sealing and narrow saw cuts.

Joint sealing: Joints with saw cuts approximately 1/4-inch wide may be sealed using bituminous material, asphalt rubber, silicone, or preformed compression joint seal materials during initial construction (depending on climate region). Sealing on a regular basis can decrease pavement damage. Joint resealing will typically be needed every 10 to 15 years depending on the material type, climate, and pavement conditions. It is often performed along with other pavement preservation work, including spall repair, individual slab replacement, and grinding.

Narrow saw cuts: Some states specify thin (1/8±1/16 inches) saw cuts to help prevent incompressibles from entering the joint. When incompressibles are less than 1/8-inch wide, they typically do not result in excessive stresses along the joint face and spalling.

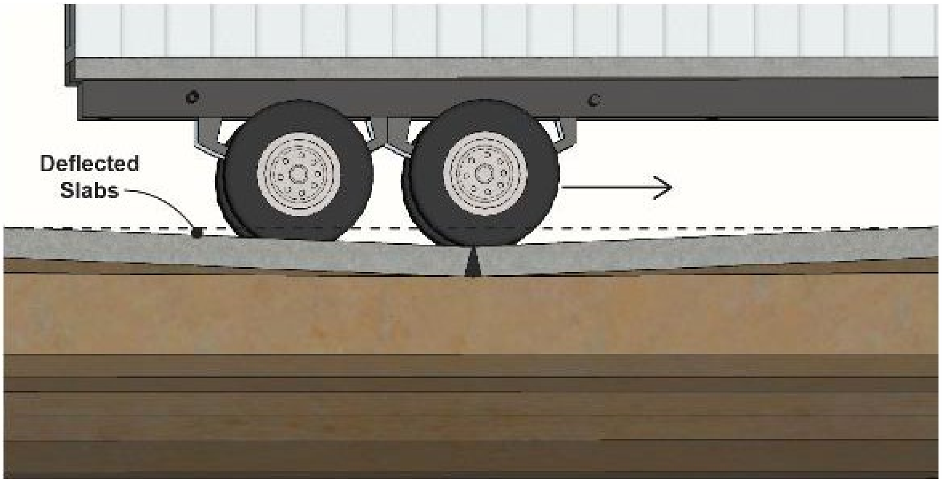
Deflection Spalling (Compression Shear)

Thinner concrete pavement slabs, such as general aviation airports or concrete overlays over asphalt, typically have higher deflection values than conventional full depth concrete pavements and therefore can have occasional deflection spalling. This type of spalling is sometimes referred to as compression shearing or compression failure. Over time, when combined with deflection and compression cycles, micro cracks can form in the concrete joint, reducing its strength and durability. After a critical number of cycles have been reached, spalling can occur.

Cause

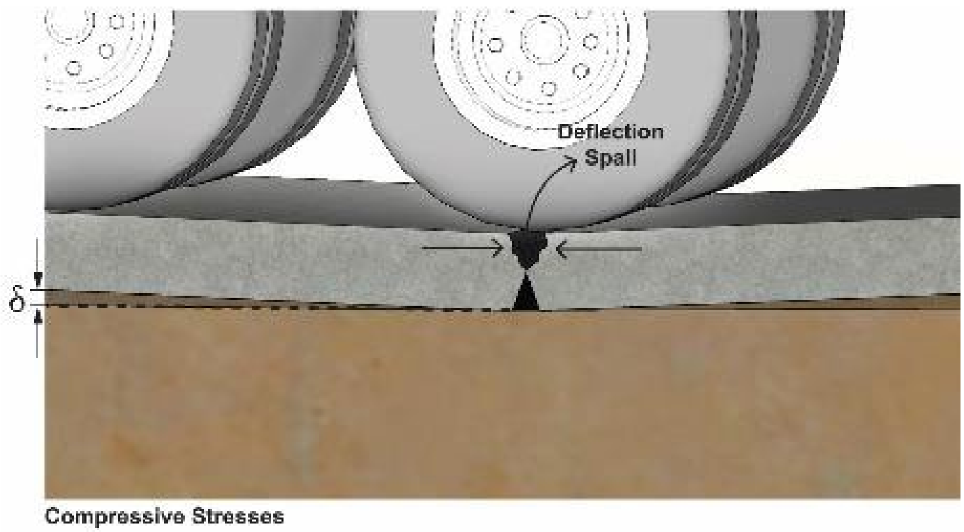
Undowelled transverse joints are deflected by heavy vertical loads. The tops of the slabs will bear on each other and the joint can crush if it is tight or filled with incompressible materials (Figure 8.12). The severity of the problem is also dependent upon the ability of the base or subbase to resist deformation due to the vertical loading.

Figure 8.12 Deflection spalling mechanism without dowel bars



a. View of deflection spalling when the ability of the base or subbase cannot resist deformation due to the vertical heavy tire loading and lack of dowel bars

After Ruiz et al. 2005 and from Taylor et al. 2007, National Concrete Pavement Technology Center



b. Close-up view of the tops of the panels bearing on each other; the joint can crush if it is tight or filled with incompressible materials

After Ruiz et al. 2005 and from Taylor et al. 2007, National Concrete Pavement Technology Center

Figure 8.13 Deflection spalling of crack due to inadequate consolidation of the subgrade

Dale Harrington, HCE Services

Subgrade softening and base compression over time and occasional weak concrete at a joint caused by inadequate consolidation during construction can cause deflection spalling. This can sometimes occur at a construction joint if low quality PCC is used to fill in the last bit of slab volume, or if dowels are improperly aligned. See Figure 8.13.

Prevention

If the joint/crack width is kept tight and the incompressibles are minimized, deflection spalling can be prevented. The tight joint crack will maintain the load transfer and the deflections will be reduced. Dowel bars also substantially reduce the potential for this type of spalling. To keep the joint tight use a subbase with low deformation properties such as a stabilized subbase. Also, proper consolidation of the concrete pavements provides proper strength of the joint face.

Joint Sawing Raveling

There is a window of time during construction to saw contraction joints in new concrete pavements. The window begins when concrete strength is sufficient for sawing without raveling along the cut and ends before random cracking starts. Figure 8.14 depicts sawing too early, which can cause unacceptable raveling by breaking the paste/aggregate bond to the depth of the saw cut.

Figure 8.14 Unacceptable joint raveling from sawing too early

ACPA

Cause

Sawing too early can cause the saw blade to pull aggregate particles free from the pavement surface along the cut. The resulting jagged rough edge leads to water and deicing chemical entrapment, which then leads to spalling. Some raveling is acceptable, especially when a second saw cut is made for joint sealer, such as, longitudinal joints when early entry saw is initially used.

Prevention

The saw operator may scratch test the concrete surface to get a good indication of when the sawing can commence. Ensure that the sawing equipment is well maintained and that the appropriate blades are sharp and are suitable for the aggregate in the mixture. A software tool known as HIPERPAV (http://www.hiperpav.com/software/) can be used to evaluate the strength and stress development of the pavement to help predict the sawing window. When the information is available designers and contractors can input maturity strength parameters into HIPERPAV, to calculate a saw cutting time window. Maturity data accurately represents strength gain in early-age concrete.

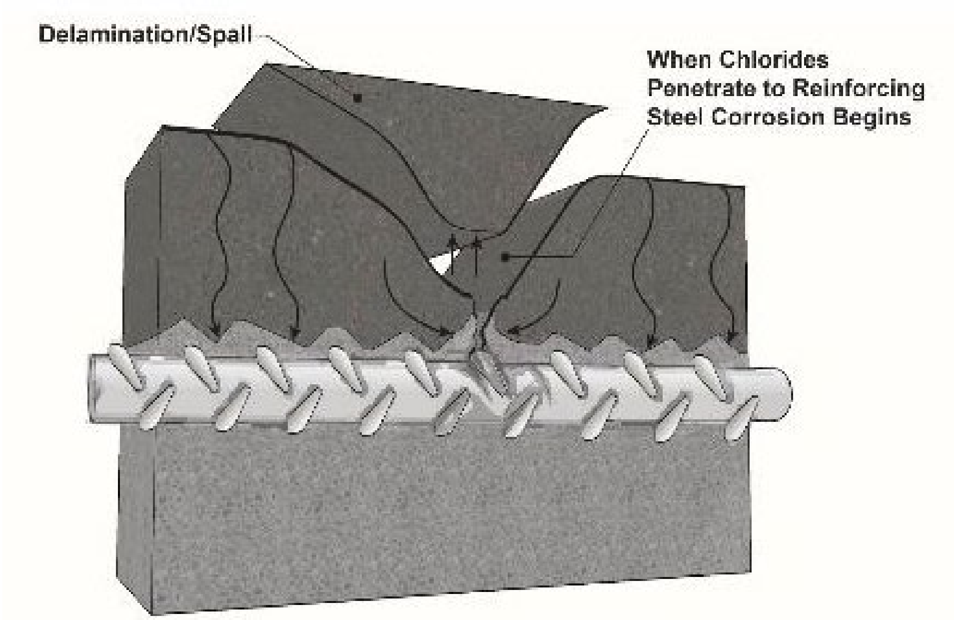
Chloride Penetration

Steel is used in dowel and tie bars, which when corroded, can lead to spalling. Steel, like most metals, is thermodynamically unstable under normal atmospheric conditions and will release energy and revert back to its natural state—iron oxide, or rust. This process is called corrosion. No other contaminant is documented as extensively in literature as a cause of corrosion of metals in concrete than chloride ions. The mechanism by which chlorides promote corrosion is not entirely understood but the most popular theory is that chloride ions penetrate the protective oxide film easier than other ions, leaving the steel vulnerable to corrosion.

Cause

Sodium, calcium, and magnesium chlorides are introduced into the concrete from the application of deicing salts. Penetration of chloride starts at the surface and moves inward. Penetration does take time and depends on the amount of chlorides coming in contact with the concrete, the permeability of the concrete, and the moisture present. If steel reinforcing in the concrete has no corrosion coating or has defective coatings, eventually the concentration of chlorides on the reinforcing steel will cause corrosion when moisture and oxygen are present. This will result in spalling as shown in Figure 8.15.

Figure 8.15 Spalling of reinforcing steel due to applications of chlorides from deicing salt



a. Cross-section of concrete with a textured tie bar with some corrosion embedded in the concrete
John Donahue, Missouri DOT

b. Close-up view of corrosion of reinforcing steel causing spalling
John Donahue, Missouri DOT

The risk of corrosion increases as the chloride content of the concrete increases. When the chloride content at the surface of the steel exceeds a certain limit, called the threshold value, corrosion will occur if water and oxygen are also available. Although chlorides are directly responsible for the initiation of corrosion, they appear to only play an indirect role in the rate of corrosion after initiation. The primary rate-controlling factors are the availability of oxygen, the electrical resistivity and relative humidity of the concrete, and the pH and temperature. High pH in concrete (usually greater than 12.5) initiates a passive protective oxide film to form on steel. However, the presence of chloride ions from deicers or seawater can destroy or penetrate the film. Once the chloride corrosion threshold is reached, an electrochemical current is formed along the steel or between steel bars and the process of corrosion begins.

Prevention

Corrosion of embedded metals in concrete can be greatly reduced by placing concrete pavements with low permeability (low w/cm ratio of about 0.40) and sufficient concrete cover. Lower-permeability concrete can be helped with the use of fly ash and slag. Supplementary cementitious materials also increase the concrete resistivity, thus reducing the corrosion rate even after it initiates. Additional measures to mitigate corrosion of steel reinforcement in concrete include the use of corrosion inhibiting admixtures, coating of reinforcement (for example, with an epoxy resin), and use of sealers and membranes on the concrete surface. Sealers and membranes, if used, have to be periodically reapplied.

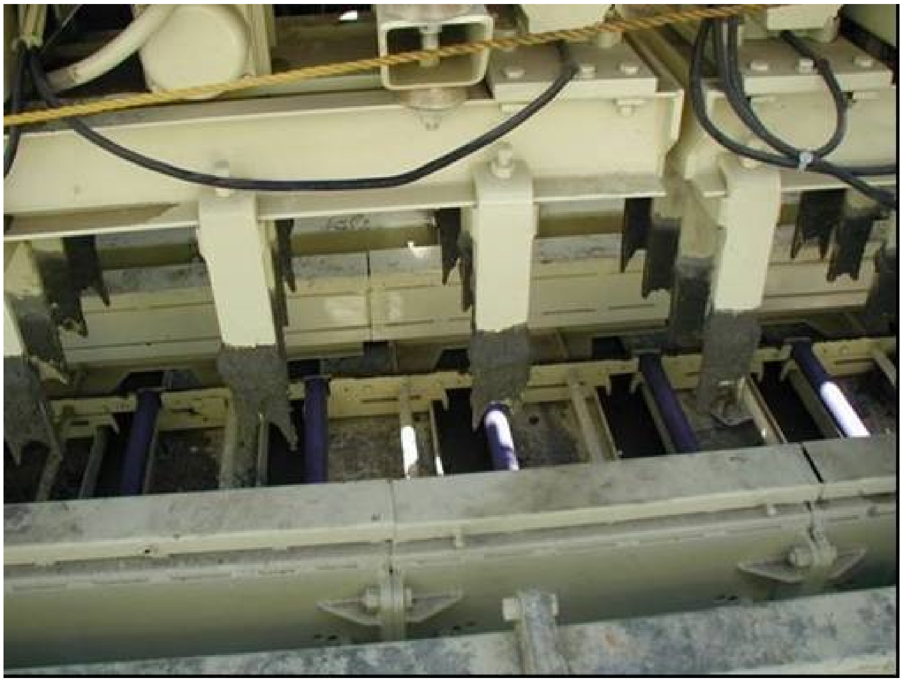
Dowel Bar Misalignment

Dowel bars for load transfer between panels need to be placed parallel to the pavement surface and to the longitudinal joint to enable free, uninhibited opening and closing of the joints. In current practice, the dowel bars are placed using either pre-fabricated dowel baskets or dowel bar inserter (DBI); see Figure 8.16.

Figure 8.16 Placement of dowels

a. Dowel bar baskets for a transverse joint and tie bar baskets for a longitudinal joint

Hoegh and Khazanovich 2009, University of Minnesota



b. Close-up of the dowel bar inserter on a paving machine
Hoegh and Khazanovich 2009, University of Minnesota

The dowel bars should also be placed centered on the joint to ensure adequate embedment in both approach and leave slabs for load transfer. To ensure adequate concrete cover at slab, the bars should be placed near mid-depth of the pavement. The position of the bars along the joint is also important to ensure the bars are placed where they are needed to provide load transfer. According to FHWA any measurable deviation in dowel bar position from the ideal position may be defined as misalignment or misplacement

Cause

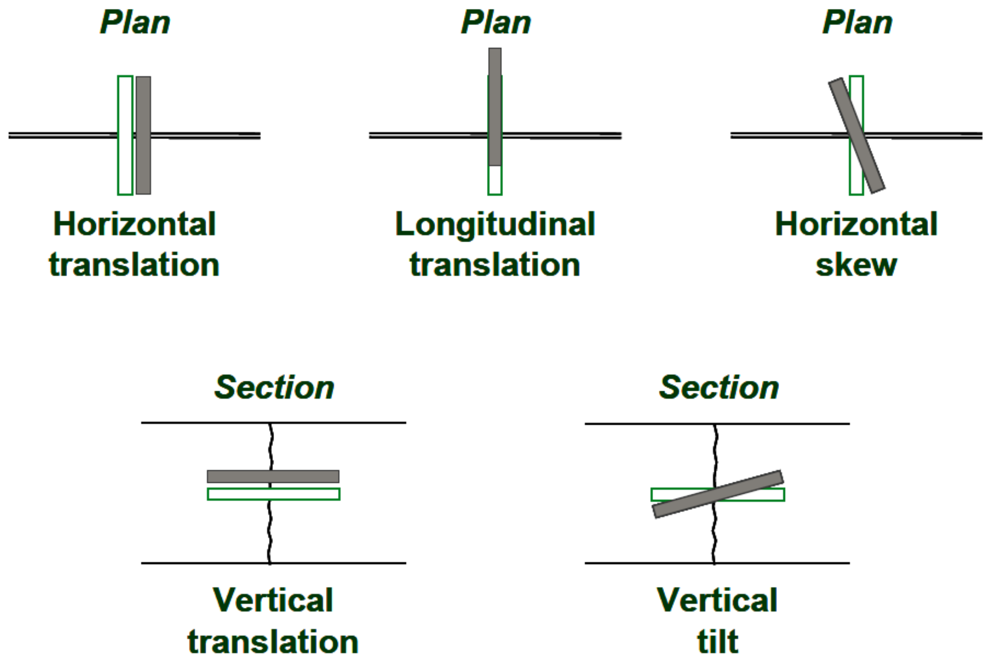
Proper placement: When dowel bars are not properly placed causing misalignment, both horizontally and vertically, the dowel bar(s) can lock up the joint, causing compression in the joint through concrete contraction. Poorly anchored dowel bar assemblies or incorrectly used BDIs cause misaligned dowels during concrete placement, resulting in concrete spalling, as shown in Figure 8.17. Similar spalling, as shown in Figure 8.17, can occur with improper construction practice of placing concrete earlier on dowel bar cages prior to paving to prevent them from moving when the mass of concrete from the paver is pushed over the cages.

Figure 8.17 Spalling from dowel bar misalignment

Jeff Uhlmeyer, Washington State DOT

Figure 8.18 illustrates the possible types of dowel misalignments identified by (Tayabji 1986).

Figure 8.18 Types of dowel bar misalignment



After Tayabji 1986 and Yu and Tayabji 2007 (from Hoegh and Khazanovich 2009)

The typical specification in the U.S. for longitudinal translation (or side shift) and vertical translation is 1 inch, whereas the requirement on horizontal skew or vertical tilt misalignments is 0.25 to 0.375 inches for 18-inch long dowel bars. The typical joint movement for 15-foot slabs is 0.08 inches to 0.12 inches, even in colder areas. In a recent study conducted by Michigan State University (Prabhu et al. 2006) displacements in excess of 0.67 inches were needed to produce spalling or cracking. The findings from the Michigan State University study (Prabhu et al. 2006) verified that the number of misaligned bars affects the pull-out force: the greater the number of misaligned bars present, the higher the force (per bar) needed to open the joint. The findings of laboratory studies suggest that both the magnitude of misalignment and the number of misaligned bars present at a joint may affect the potential for joint locking (as indicated by increased pull-out force). See *National Cooperative Highway Research Program (NCHRP) Report 637: Guidelines for Dowel Alignment in Concrete Pavements* (Khazanovich et al. 2009) for further details.

The potential impacts of various types of dowel misalignment on pavement performance, as identified by Tayabji (1986), are summarized in Table 8.3.

Table 8.3 Effect of each type of misalignment

Type of Misalignment	Effect on Spalling	Cracking	Load Transfer
Horizontal translation	No	No	Yes
Longitudinal translation	No	No	Yes
Vertical translation	Yes	No	Yes
Horizontal skew	Yes	Yes	Yes
Vertical tilt	Yes	Yes	Yes

Source: Tayabji 1986

Improper Expansion Cap Placement: When expansion caps at end of the dowel bars are not installed with 1/4-inch minimum clearance from the end of bar (to allow for slab contraction and expansion), spalling of the slab may occur due to expansion pressure from the end of bar against the concrete. (See Figure 8.19.)

Figure 8.19 Dowel bar expansion cap

a. View of expansion cap on dowel bar
Randy Riley, Illinois Chapter, Inc. – American Concrete Paving Association

b. View of an improper placement of an expansion cap on the end of a dowel bar resulting in expansive pressure on the concrete causing spalling

Randy Riley, Illinois Chapter, Inc. – American Concrete Paving Association

Shallow Placement of Dowel Bars: When the dowel bars are placed shallow and without adequate cover, the movement of the concrete slab can cause spalling. (See Figure 8.20.)

Figure 8.20 Spalling from high dowel bar placement in the concrete slab

John Donahue, Missouri DOT

Dowels Placed Too Close to the Edge of the Joints: Another cause of spalling, particularly at the intersection of joints, is when the dowel is too close to the edge of the joint (typically less than 6 inches) and/or there is interference between the tie and dowel bars. As shown in Figure 8.21, this interference occurs when the tie bars are automatically inserted in the centerline joint and the spacing is gradually changed enough during forward movements of the paving train to put the tie bars directly over the end of the dowel bar basket.

Figure 8.21 Dowel bar too close the edge of the longitudinal joint and interference of the tie bar

Dan DeGraaf, Michigan Concrete Association

Prevention

Design and construction factors can have a significant effect on dowel alignment achieved in concrete pavements. A number of broad factors include the following:

- Care during basket transportation and placement.
- Concrete placement practice: the overloading of concrete head in front of the paver must be avoided. Some contractors find that placing concrete over the dowel assembly before passage of the paving machine minimizes dowel assembly pressure affects.
- Align dowels to be centered over the joint line and the joint sawed correctly.
- Handling, placement, and anchoring of dowel baskets.
- Proper equipment type, adjustments, and operator.
- Basket rigidity: cutting basket support wires in dowel basket assembly can lead to twisting and movement of the basket.
- Paving operations: adjustments may need to be made to the paver to avoid the paving pan dislodging dowel baskets from their anchors. This is particularly true when paving thickness is less than 8-inches thick.
- Ensure adequate cover over dowel bars.

- Prevent dowels from being too close edge of longitudinal joint. Consider moving last dowel 8 inches, rather than 6 inches from the edge.
- Location of saw cut over implanted dowels.
- Field inspection during construction: On-site scanning devices can be used periodically during paving operations to check the location of dowel bars. Magnetic imaging tomography (MIT) scans and ultrasonic tomography scans are such devices. Ground penetrating radar (GPR) can be also used after the paving is complete to check specification requirements.

To prevent movement of the baskets, the most critical factor is securely anchoring the dowel bar basket assemblies prior to paving so that the concrete placement will not disturb the dowel bar alignment. Prefabricated dowel baskets are placed on a prepared base at the planned joint locations, and typically anchored using nails (for stabilized subbase or existing pavement) or stakes (for granular subbase) prior to paving. For 12- to 14-foot lanes, the National Concrete Pavement Technology Center suggests anchoring the bottom rail of the dowel baskets securely with a minimum of eight stakes per basket with four stakes installed on both basket's horizontal support legs.

Some contractors install pins on both horizontal leg supports for every two dowels (see Figure 8.22).

FHWA Technical Advisory T5040.30, *Concrete Pavement Joints,* recommends securing dowel baskets to the base with steel stakes having a minimum diameter of 0.3 inches and embedding the stakes in the base a minimum depth of 4 inches for stabilized dense bases, 6 inches for treated permeable bases, and 10 inches for untreated permeable bases, aggregate bases, or natural subgrade.

Proper Anchoring of Dowel Bar Baskets: The final anchoring configuration selected must assure that the dowel basket assemblies do not move during concrete placement. It is suggested that the contractor develop a quality control plan to address the anchoring strategy for review by the engineer.

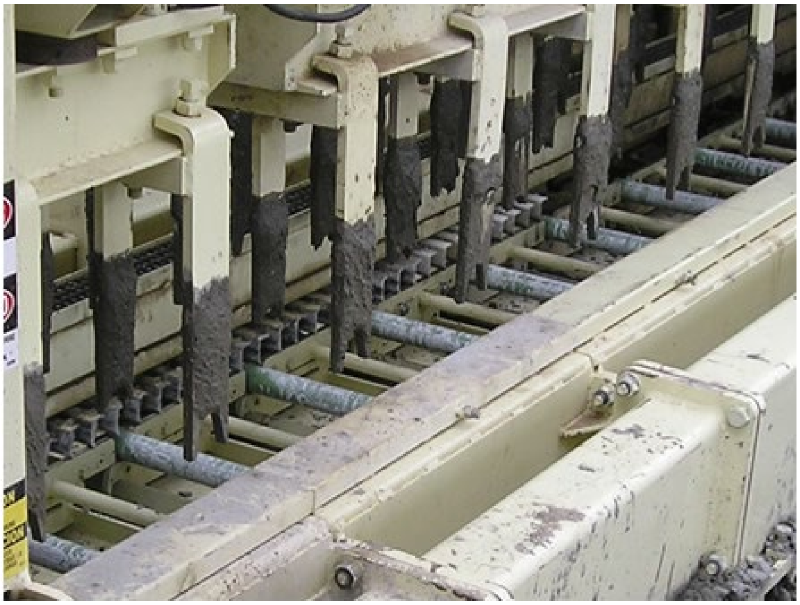
Figure 8.22 Typical dowel bar assembly with proper anchoring of the basket

a. Dowel bar assembly
Privrat 2012

b. Anchor clip for dowel bar anchoring
Eric Ferrebee, American Concrete Paving Association

The 1990 Dowel Bar Inserters: Over the last 15 years, DBIs have shown good results for the placement of dowel bars. Well-graded mixes are important for the accurate placement of the dowel bar with DBI. Gap-graded mixes should be avoided. Also, misalignments can result from improper equipment adjustments, as well as any problems with the concrete mix. The number of forks and vibrating frequency affect dowel alignment, as well as, consolidation of concrete around and above the dowel bars. As Hoegh and Khazanovich (2009) reported from personal communication with Ron Guntert in 2004, “the dowel bars must act as the vibrator to move and consolidate the concrete in its path around the dowel bar as the bar is inserted into the concrete” (Figure 8.23).

Figure 8.23 Dowel bar inserters (DBIs)



Guntert and Zimmerman Const. Div., Inc.

Table 8.4 provides a summary of the mechanisms leading to spalling as well as factors that can be used to help prevent or mitigate it.

Table 8.4 Summary of causes and prevention or mitigation

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Freeze-Thaw Damage	Saturation of concrete at the joint/crack Poor air void system Marginal aggregate soundness Use of calcium or magnesium chlorides for deicing causing damaging calcium oxychloride	Provide sufficient drainage of subbase Ensure joints crack through Seal joints in pavements with marginal drains or specify thin joints with drainable base Eliminate use of backer rod (cold weather states)	Require adequate air entrainment admixtures to stabilize air voids Specify low w/cm ratio (<0.42) to reduce permeability Specify well graded, durable and low CTE aggregate Use SCMs to help lower permeability and reduce risk of oxychloride reaction Consider use of penetrating sealants over the joints	Check air quantity in front and periodically behind paver and maintain at least 5% air content behind the paver Ensure use of high quality curing compound Ensure proper application and placement timing of curing compound Where appropriate use SAM to measure both the air void spacing and air volume of plastic concrete	Use sodium chlorides for deicing chemicals where sufficient for safety Employ a minimum 30 day drying before applying deicing chemicals Ensure proper maintenance of sealants in joints Conduct periodic inspection and maintenance of subdrain outlets Look for early detection of spalling and use of PDRs

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Joint Compression Damage	Incompressibles in joints/cracks Poorly sealed or unsealed joints Compression shear from deflection of slab, heavy truck traffic, lack of load transfer across the joint, or poor subgrade support Inadequate concrete consolidation High CTE of aggregates	Require low deformation properties in subbase Provide proper load transfer design across joints Specify low deformation properties of the subbase to minimize deflection	Choose aggregates with low CTE	Ensure proper sealing of joints Proper concrete consolidation at joints	Use proper panel removal methods to prevent adjacent slab damage at the joint Adequate panel replacement by providing relief cuts
Early Sawing Damage	Sawing joints too early chips or frays the edges of the joint	Provide proper specifications for saw joint timing	N/A	Conduct trial saw sections to prevent raveling	Provide proper maintenance of sealants in joints

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Chloride Penetration	Deicing salts with high permeability concrete High reinforcing steel High relative humidity of concrete	Enforce cover requirements of embedded steel Specify coating of reinforcement	Specify low w/cm ratio Use of SCMs to reduce chloride transport Require corrosion inhibiting admixtures	Ensure proper placement of embedded steel	Monitor steel for early signs of corrosion and repair early
Bar Misalignment	Poor alignment of dowel and tie bars Improper fastening of dowel bar baskets Overloading of the head of concrete during placement Poor basket rigidity due to cutting basket support wires	Adequate basket fastening specifications Adequate coating of dowel bars specification	N/A	Ensure proper alignment and cover of bars Provide proper fastening of basket assemblies Minimize head of concrete in front of machine Prevent cutting basket support wires Prevent dowel too close to longitudinal joint	Replace misaligned dowel bar and bars with lack of cover Repair locked up dowel bars too close to longitudinal joint

6. Treatment and Repairs

When spalling is first observed in transverse and/or longitudinal joints, the evaluation of what's causing the spalling and the rate of deterioration is critical in the selection and timing of the most cost-effective repairs. The proper evaluation and timing of repairs are the two most important elements of maintaining concrete pavement at the highest service level and at the lowest cost. Asset management's cornerstone statement of "selecting the right treatment, at the right time, at the right cost" definitely applies to spalling. This is especially accurate since spalling can move from a relatively low cost repair to a major cost repair in a rather short time period.

The National Concrete Pavement Technology Center's second edition of the *Concrete Pavement Preservation Guide* (Smith et al. 2014) provides valuable guidance and information on the selection, design, and construction of cost-effective concrete pavement preservation treatments such as spalling. The following is a summary of timing, treatment, and repairs for concrete pavement spalling in transverse and longitudinal joints and cracks.

Repairs of Spalling

Partial-Depth Repairs (PDRs)

PDRs are a time-tested method for spalling repairs. The repair methodology, materials, and equipment have substantially improved over the last 15 years. When applied at the appropriate locations, PDRs have proven to be more cost-effective than full-depth repairs (see Figure 8.24).

Figure 8.24 Partial-depth repair (PDR)

a. Milled joints for removal of spalling for PDR
National Concrete Pavement Technology Center

b. Finished partial-depth repair (PDR)
Kevin McMullen, Wisconsin Concrete Pavement Association

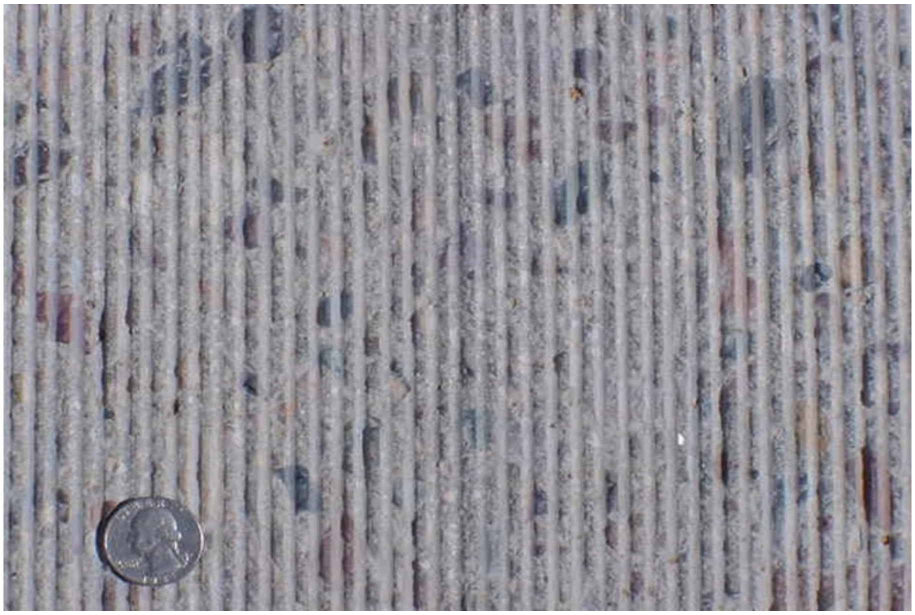
The primary limitation of using PDRs is the depth of the repair. All PDR should be in the top one-half of the pavement depth to have the intended service life of 10 to 15 years.

In cold weather states where joint or crack spalling reaches the low severity level (less than 3 inches wide measured to the face of the joint), in a period of 10 to 15 years, the spalling is typically a result of saturated joints with a moderate to poor air void system. Typically, with saturated joints, shadowing of the joints does appear. In order to be cost effective, it is recommended that plans be made for proper draining of the joints, removal of any backer rods in the joint, and partial-depth repairs begin as soon as possible and must be completed within a 2-year window. When spalling reaches a moderate level of 3 to 6 inches wide (from the face of the joint), the severity has reached a point requiring immediate PDRs. Even in low severity levels, cores should be taken to determine if the deterioration is in the top one-third to one-half of the pavement thickness. In some situations, spalling has extended below the limits of PDR, and full depth repairs would be required. The reestablishment of the pavement joints at the same opening dimension as the in-place adjacent joints is very important. This prevents point loading and compression failures of the repairs.

If measurable spalling is evident in pavements less than 7 years old and the concrete mortar in the joints show evidence of flaking, a petrographic analysis should be done to determine the air void spacing and volume in the joints. If the air void system is poor, and is unable to drain the saturated joints, then PDR is not the proper repair. Historically pavements in this condition and age, with PDRs, show continued deterioration adjacent to the PDR. FDRs would be required under this condition, and even then they may have a limited service life.

Diamond grinding (Figure 8.25) should be considered after repairs are made to produce a smooth riding surface and to restore the International Roughness Index IRI. See Chapter 5 of the *Concrete Pavement Preservation Guide* (Smith et al. 2014) for proper partial-depth repairs.

Figure 8.25 View of diamond ground concrete pavement surface



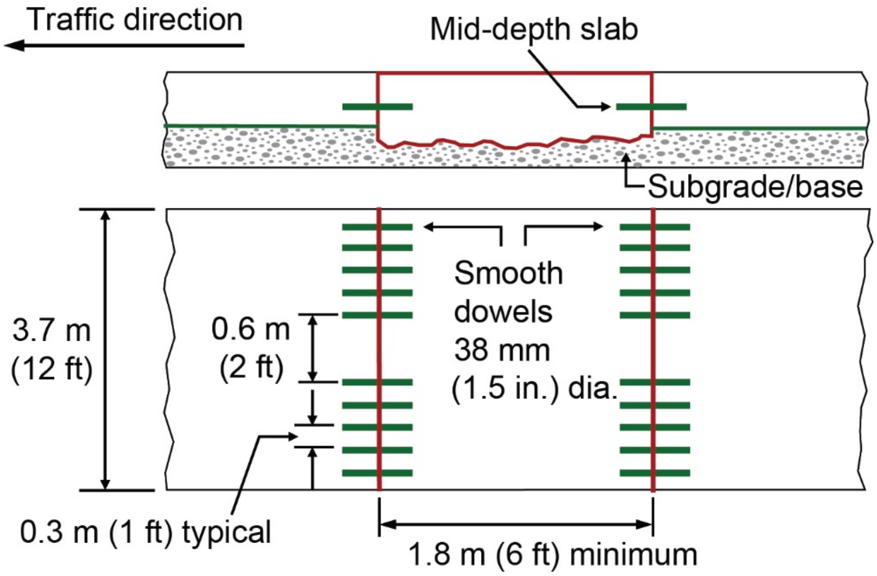
Larry Scofield, American Concrete Pavement Association

Full-Depth Repairs (FDRs)

When spalling has reached a width greater than 6 inches measured to the face of the joint, the chance of the depth of the deterioration exceeding half the pavement thickness is much greater. When this occurs, PDR is typically not a long-term repair as compared to FDR.

FDR of the joints is a good candidate to handle higher severity levels of joint and crack spalling (see Figure 8.26).

Figure 8.26 Example of full-depth repair of a spalled joint (plan view)



After Smith et al. 2014, National Concrete Pavement Technology Center

Although FDR can be designed and constructed to provide good long-term performance, the performance of FDRs is very much dependent on appropriate application and use of effective design and construction practices. Many 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 far deteriorated. See Chapter 6 of the *Concrete Pavement Preservation Guide* (Smith et al. 2014) for proper FDRs. Cost efficiency of full depth repairs depends a great deal on the frequency of transverse and longitudinal joint spalling. For example, if a pavement section has transverse joints not requiring every joint to be repaired, FDR is a strong candidate. However, if the pavement section has nearly every joint requiring FDR, it is recommended that a cost analysis be completed to determine if it is better to replace full panels or to construct an unbonded concrete overlay as a rehabilitation method.

Unbonded Concrete Overlay on Concrete (UBCOC)

UBCOC would be a proper minor rehabilitation technique for a concrete pavement with high severity spalling (Figure 8.27).

Figure 8.27 Spalling and overlay (before and after)

a. Severe spalling on original concrete pavement (before overlay)
Todd LaTorella, Missouri/Kansas Chapter, American Concrete Paving Association

b. After placement of an unbonded concrete overlay
Todd LaTorella Missouri/Kansas Chapter, American Concrete Pavement Association

The deteriorated joints can be milled to remove loose material and backfill with lower quality concrete or flowable mortar. This eliminates expensive repairs since the joints do not have to be reestablished since reflective cracking is not an issue with properly designed and constructed unbonded overlays.

A separation layer consisting of either a nonwoven geotextile fabric or thin HMA interlayer is placed between the existing concrete pavement and the new unbonded concrete overlay. The separation layer prevents bonding and thus eliminates reflective cracking. See the National Concrete Pavement Technology Center's *Guide to Concrete Overlays* (Harrington and Fick 2014) for additional details.

Maintaining Pavement

Winter Maintenance

Winter maintenance activities to remove snow and ice on highway pavements include some sanding but in recent years, the use of anti-icing solutions are more common. Some agencies have applied the following approaches to minimize deicing effects to the concrete joints/cracks.

- Currently research is being conducted on several roadways and highways in Iowa by the National Concrete Pavement Technology Center to determine the effectiveness of surface sealants in and over the joint to reduce immigration of water and deicing solutions into the concrete.
- Use of sodium chloride (NaCl) brines rather than magnesium chlorides ($MgCl_2$) or calcium chlorides ($CaCl_2$).
- Some cities have employed a minimum curing period on the residential streets before applying any deicing chemicals to new concrete.

Maintaining Sealed Joints

Unless the pavement is designed for unsealed joints, all sealed concrete joints should be periodically checked (approximately every 2 years) to determine if they need to be resealed or refilled. The purpose of joint and crack sealing is to keep the incompressibles and moisture out of the joints. Joint fillers will have to be replaced at certain intervals, depending on type of filler used (liquid form or preformed). The performance of the joint and crack sealing treatments (i.e., how long they effectively perform their primary functions) varies considerably with the type of material, the reservoir design, prevailing climatic conditions, and the quality of the installation process.

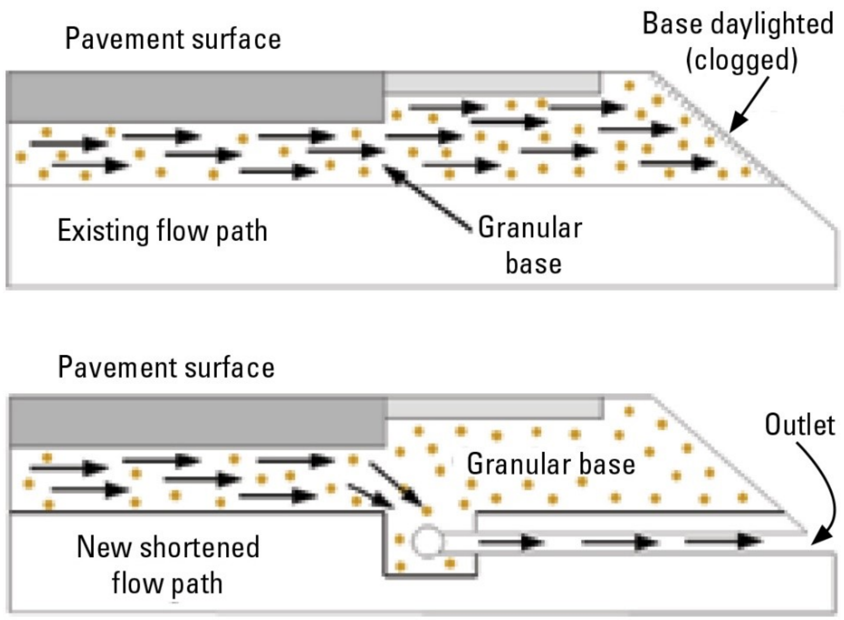
Based on a review of a number of available studies, the performance of concrete joint resealing installations was noted to range from 2 to 8 years, while the performance of concrete crack sealing was noted to range from 4 to 7 years (Peshkin et al. 2011). These are based on a failure definition of 25% of the sealant installation being no longer functional. However, longer performance lives are possible. For example, using nearly 7 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). A substantial portion of water that enters a pavement from the surface does occur through the longitudinal joints or cracks. The amount can be as much as 65% to 80% and typically enters through the lane/ shoulder joint. It is for this reason special attention needs to be given to maintaining the sealants of these joints.

Maintaining Subdrain Systems or Installing Retrofit Drainage Systems

The proper removal of water from the concrete pavement joint system is an important aspect of preventing joint spalling, particularly in cold-weather states. A recent study in Iowa found that a significant percentage 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 periodic inspection and cleaning of the drainage structure inlets and outlets are performed at approximately 2-year intervals.

When an existing pavement begins showing signs of spalling due to moisture, a jurisdiction has the choice to retrofit an existing pavement with an edge drained system. As shown in Figure 8.28, one method is to shorten the drainage path to an outlet, particularly when an existing subdrain system is plugged or there is an impervious earth blockage in the shoulder area.

Figure 8.28 Example of shortening drainage path to an outlet



Kurt Smith, Applied Pavement Technology, Inc.

The shortened drainage path is accomplished by tying an aggregate outlet ("french" drain) into the existing granular subbase and daylighting it out to the shoulder foreslope area or installing a pipe edge drain and outletting it to a low point in the profile grade to the foreslope or ditch.

The national performance of pavements with retrofit 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 at the time of retrofitted edge drains. For proper selection in installation of retrofit edge drains, see the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

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CHAPTER 9. FAULTING

1. Description

Faulting is the difference in elevation across a joint or crack in a pavement due to loss of load transfer. It is a symptom of the loss of uniform base or subgrade support, normally from the lack of dowel bars (Snyder 2011).

Figure 9.1 Concrete pavement joint faulting

National Concrete Pavement Technology Center

A number of factors can occur that initiate concrete pavement faulting, including truck traffic loadings or water getting trapped in the base layers. When water is trapped in the base layers, it initiates a pumping action that expels or rearranges base/subgrade material at joints and cracks, resulting in void spaces with no support.

Faulting may occur due to differential base support, which can result from nonuniform base materials, secondary compaction/settlement of open-graded subbase materials, or intermingling of subgrade soils with open graded subbases. It may also occur due to loss of load transfer from corrosion of the dowel system, panel movement causing loss of contact between non-doweled panels (Khazanovitch et al. 2009) , or from aggregate toughness (weaker coarse aggregate materials breakdown easily, placing a greater reliance on base support for load transfer).

Faulting increases with time and continues to degrade the ride while dynamic loading from traffic initiates cracking and spalling of the concrete pavement joints/cracks.

Faulting is typically visible when the user looks in the rear-view mirror and observes the vertical face of the pavement joints (positive faulting; see Figure 9.1). It is also possible to see the opposite phenomena (negative faulting) on uphill sections of pavements.

Faulting may also exist along longitudinal joints and cracks. Severe faulting of joints and cracks may require major action, such as corrective subgrade work and complete panel replacement. Longitudinal faulting may create a safety issue, especially with motorcyclists.

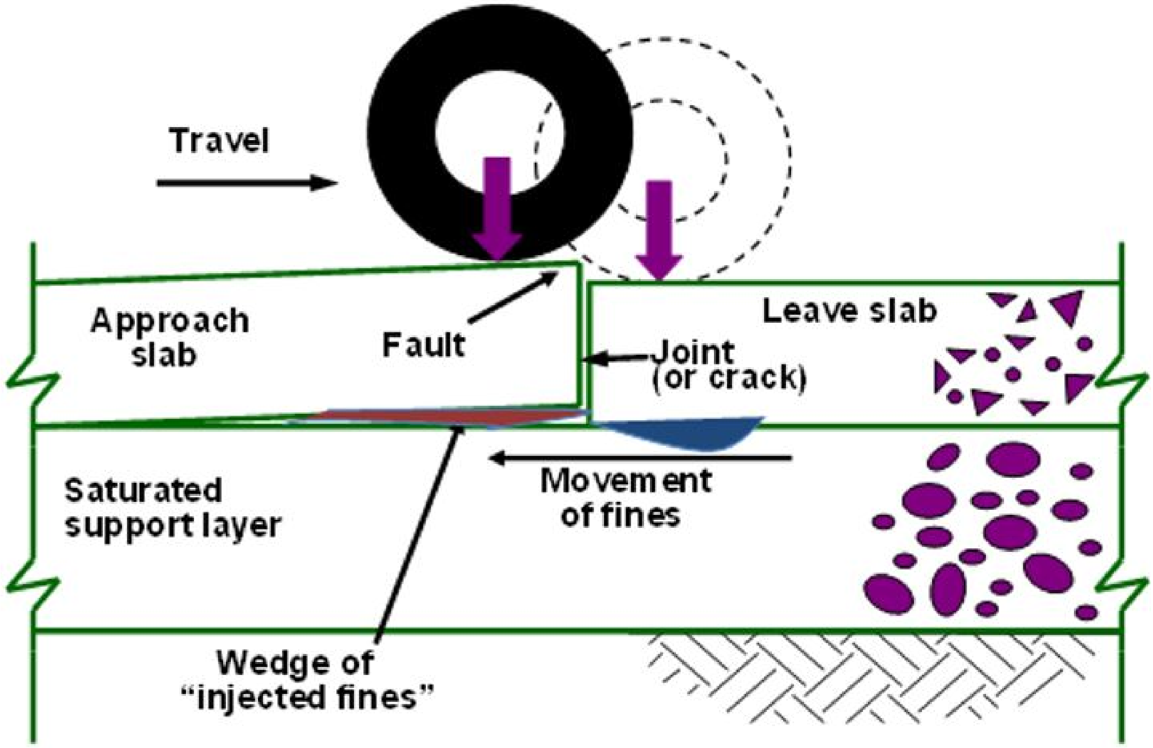
Note: The evaluation of a pavement faulting issue must start with a subbase and subgrade condition study.

Pavements constructed on a dense-graded base or directly on cohesive soils have a higher potential for pumping and development of voids. Additionally, after many years of service, these pavements may have deformed the soil base beneath them so that while they are faulted they still maintain complete contact with the grade below. The key is to ensure that the pavement has complete contact with the grade below prior to initiating pavement repair.

In a quest to provide positive drainage, many DOTs have experimented with drainable base layers. In some states, highly drainable aggregate layers were utilized. One problem with these drainable bases is that fine-grained soils infill the open-graded layers, creating significant void spaces below the pavement, particularly on undowelled pavements (See Figure 9.2). In other instances, these open-graded layers collapsed into themselves as a result of traffic loading and system vibration. Unfortunately, the only viable long-term solution for this issue is a complete replacement of the base and pavement.

The ideal base material is one that is granular and stable in nature (you can drive on it) but one that will also allow water to pass through it at a minimum rate so that a pumping pressure is never reached.

Figure 9.2 Faulting mechanism



After National Highway Insititute 1993

2. Severity

Severity levels for faulting are not applicable according to the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014); Table 9.1 is a modification of faulting levels from the Ohio DOT.

Table 9.1 Summary of fault severity and extent level of distress

Distress	Description	Severity Levels	Extent Levels	Measurement
Faulting	Difference of elevation across a joint or a crack	**Low**: Less than or equal to 1/8 in. (3 mm) fault **Moderate**: Greater than 1/8 to 3/8 in. (3 mm to 9.5 mm) fault **High**: Greater than 3/8 in. (9.5 mm) fault	**Occasional**: Faulting occurs along less than 20% of the joints and cracks **Frequent**: Faulting occurs along 20 to 50% of the joints and cracks **Extensive**: Greater than 50% of the joints and cracks are faulted	Faulting is recorded to the nearest millimeter Measurements are taken 9 ft from the centerline The average of three measurements is taken

Source: Modified from the Ohio DOT

3. Testing

Field tests used for faulting are briefly described below. More detailed information regarding testing can be found in Chapter 19.

Field Tests

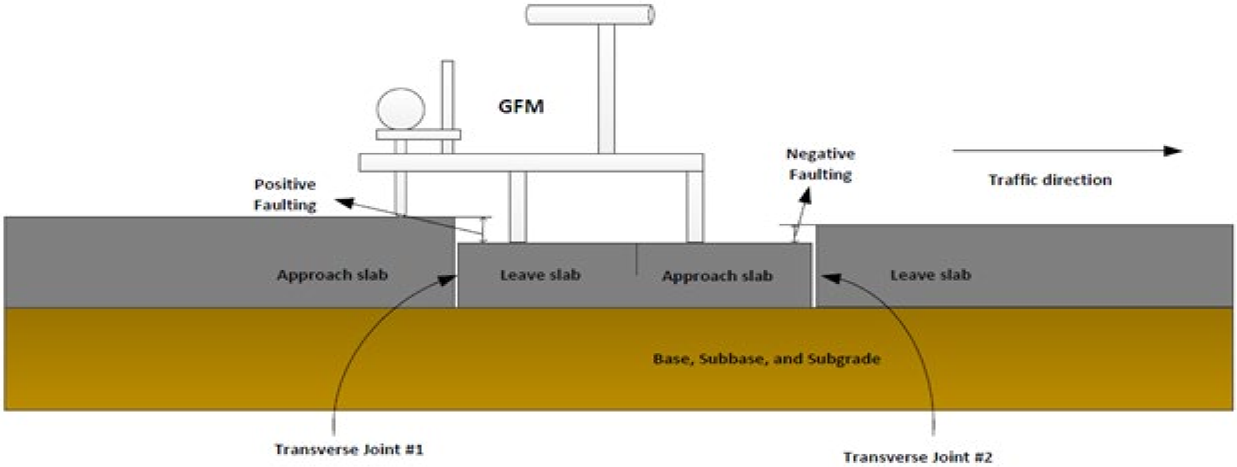
Manual – Using the Georgia Fault Meter

The Georgia Fault Meter (Figure 9.3a and b) electronically measures faulting at joints to the nearest millimeter. Faulting and the degree of faulting can be used to determine underlying issues, such as nonuniform subgrade, settlement, heaving, etc.

Figure 9.3 Georgia Fault Meter

a. Georgia Fault Meter in use
Minnesota DOT

Record the faulting in millimeters, to the nearest millimeter at a location 9 feet from the roadway centerline. At each location, take three measurements and record the approximate average of the readings. If the approach slab is higher than the departure slab, faulting is recorded as positive (+); if the approach slab is lower, faulting is recorded as negative (-). In Figure 9.3b, note the terminology for the approach and leave side of the joints as a function of traffic direction.



b. Illustration of a Georgia Fault Meter
Agurla and Lin 2015

Automatic – Using a High Speed Profiler (HSP)

This device provides a longitudinal profile along the wheel paths and provides ride quality values including faulting at joints and cracks which are good indicators of pavement performance. Using the high speed profiler (HSP) (See Figures 9.4a and b), follow the procedures described in *Long-Term Pavement Performance Automated Faulting Measurement* (Agurla and Lin).

These devices provide a longitudinal profile along the wheel paths in a pavement. They also provide ride quality information, including faulting at joints and cracks, which are good indicators of pavement performance. Since the HSP operates at highway speed, the advantage of this device is there is faster data collection without the need for traffic control, lane closures, safety measures, and their inherent costs.

Figure 9.4 High speed pavement profiler

a. Front view
Minnesota DOT

b. Rear view
Minnesota DOT

4. Identification of Causes

All causes of faulting are either physical or materials related. Ultimately, faulting is the result of the loss of uniform support. Unsealed joints and cracks are conduits for water intrusion, resulting in the saturation of fine subgrade soils. Designing and constructing drainable pavement substructures is an important aspect to consider in preventing the occurrence of pumping, load transfer loss, and faulting. These are all critical issues to consider in designing a cost-effective, long-life concrete pavement.

Faulting develops when traffic loading on concrete pavement joints and cracks lead to a breakdown of load transfer from failure of aggregate interlock or doweled load transfer mechanisms. Once the load transfer is reduced, continued traffic loadings pump subgrade or subbase soils in the open joint/crack, resulting in voids.

Table 9.2 provides a summary of the physical and material causes of distress.

Table 9.2 Physical and material causes of distress

Distress	Item	Description
Faulting (Physical)	Load Transfer Loss	Failure of aggregate interlock or mechanical devices that transfer load across pavement joints and cracks
Faulting (Physical)	Cracking	Longitudinal and transverse cracks that allow water intrusion and lead to future faulting, including loss of load transfer
Faulting (Physical)	Pumping	Seeping or ejection of water beneath the pavement through joints or cracks
Faulting (Physical)	Loss of Seal Integrity	Portal for intrusion of water into the grade
Faulting (Material)	Poor Aggregate Soundness	Poor quality coarse aggregate leads to early loss of load transfer due to low shear capacity Aggregate particles deteriorate resulting in loss of support

5. Evaluation

There are many interconnected causes that result in faulting of joints and cracks in concrete pavement. After failure of load transfer from mechanical devices and/or aggregate interlock, the loss of uniform subgrade support resulting from the pumping of water rearranges subgrade fines in the joints and cracks.

Faulting is prevented by good joint/crack load transfer and spacing design, base design, and subdrainage design. All causes are exacerbated by a lack of mechanical load transfer across the joints or cracks.

Materials/Chemical Mixture Related Factors

Doweled concrete pavements may lose load transfer capacity for several reasons, including loose dowels (socketing) caused by concrete mixture material degradation or lack of consolidation, necking-down of dowel bars due to corrosion, or installing dowel bars with insufficient diameter to handle heavy loads. Expansive pressures from dowel bar corrosion can cause expansion spalling at the joints and possible panel delamination at the elevation of the dowels.

Cause

Coefficient of Thermal Expansion (CTE): Aggregates with high CTE properties cause contraction and expansion of panels due to annual and daily temperature swings. Even without dowels, rising temperatures expand the pavement panels providing load transfer but the load transfer is lost when the temperature drops. The CTE issue is a bigger problem for pavement without mechanical load transfer. Since carbonate aggregate mixtures are less thermally expansive, the designer should take this reality into consideration when a variety of coarse aggregates are available.

Prevention

Concrete Mixture: To minimize the impacts of CTE, mix design is extremely important. Poor quality concrete mixtures lead to an early loss of load transfer. CTE can be reduced by decreasing the use of nondurable aggregates (aggregates with small maximum size, high paste content). This is true for both non-doweled and doweled pavements and for plain and reinforced concrete pavements. Poor quality concrete mixtures lead to loss of aggregate interlock and socketing (loose dowels), which results in loss of load transfer. With poor quality aggregate, you may need to build a better base to assist with load capacity.

Lack of Load Transfer

Cause

Faulting develops when traffic loadings on concrete pavement joints and cracks lead to a breakdown of load transfer from mechanical devices or aggregate interlock. This shear failure initiates the beginning of the faulting process.

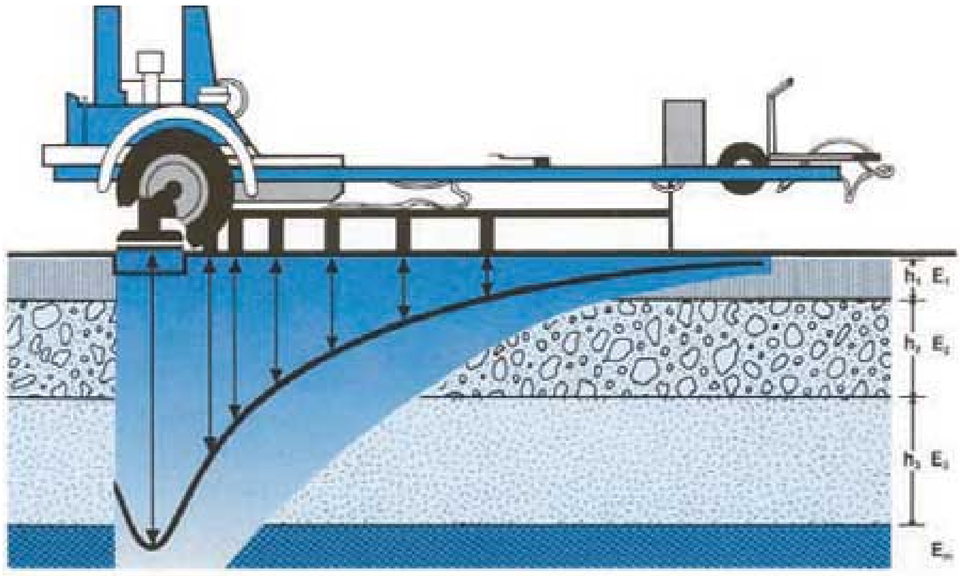
Non-doweled concrete pavement joints lose load transfer when shear friction breaks down the aggregate interlock below the saw cut in the joint. The same shear forces cause faulting of mid-panel cracks in jointed plain concrete pavement (JPCP) and in jointed reinforced concrete pavement (JRCP) after pavement reinforcement failures. Wider cracks dramatically reduce load transfer efficiency (LTE).

LTE data is needed from pavement joints and cracks to provide information for timely restorative and maintenance actions.

Measuring Load Transfer Loss

The most common method for measuring load transfer efficiency, and thus, load transfer loss (see Figure 9.5) is by using a falling weight deflectometer (FWD).

Figure 9.5 Measuring load transfer loss



Cornell Local Roads Program 2005

The device applies a load over the critical dowel, which is typically the outside dowel and measures the deflections on each side of the pavement joint. The tests are conducted only when the slab temperature is less than 70 °F to avoid expansion and closing of the pavement joints resulting in LTE not representative of typical values.

$$\text{Efficiency (\%)} = \frac{\Delta_l \times 100}{\Delta_u}$$

Δ_l = loaded side deflection

Δ_u = unloaded side deflection

This efficiency depends on several factors, including temperature (which affects joint opening), joint spacing, number and magnitude of load applications, foundation support, aggregate particle angularity, and the presence of mechanical load transfer devices.

In the analysis of test results, data may indicate low LTE but since the total deflection is small at both the approach and leave side of the joints, the risk of faulting is low. Conversely, if LTE values are high, and deflections at the joints are high, the risk of faulting is greater.

A joint with a LTE of 85 percent or less and/or a deflection difference between the panels greater than 0.13 mm (5 mils) in 5 years or less is unlikely to provide adequate long-term pavement performance (Larson and Smith 2005).

Along with visual inspection, you can use deflection data from the FWD testing to locate voids. This procedure is explained on pp. 3–5 of the American Concrete Pavement Association (ACPA)'s publication *Slab Stabilization Guidelines for Concrete Pavements*. Three FWD drop height deflections are graphed against their respective loads for the purpose of detecting voids under the slab edge. This is a very common and effective FWD analysis tool.

Prevention

Experience has proven the advocacy of using mechanical load transfer devices for concrete pavements on grade for thicknesses greater than 7 inches. In the long term, non-doweled pavements are not cost effective because they have shorter expected pavement life and they require considerable maintenance. High truck traffic increases both the design thickness and the need for load transfer devices.

Cracking

Cause

Transverse and longitudinal cracking provides an opening in the concrete which allows for water to enter the subgrade resulting in water being trapped and initiating faulting if the subgrade and subbase is erodible.

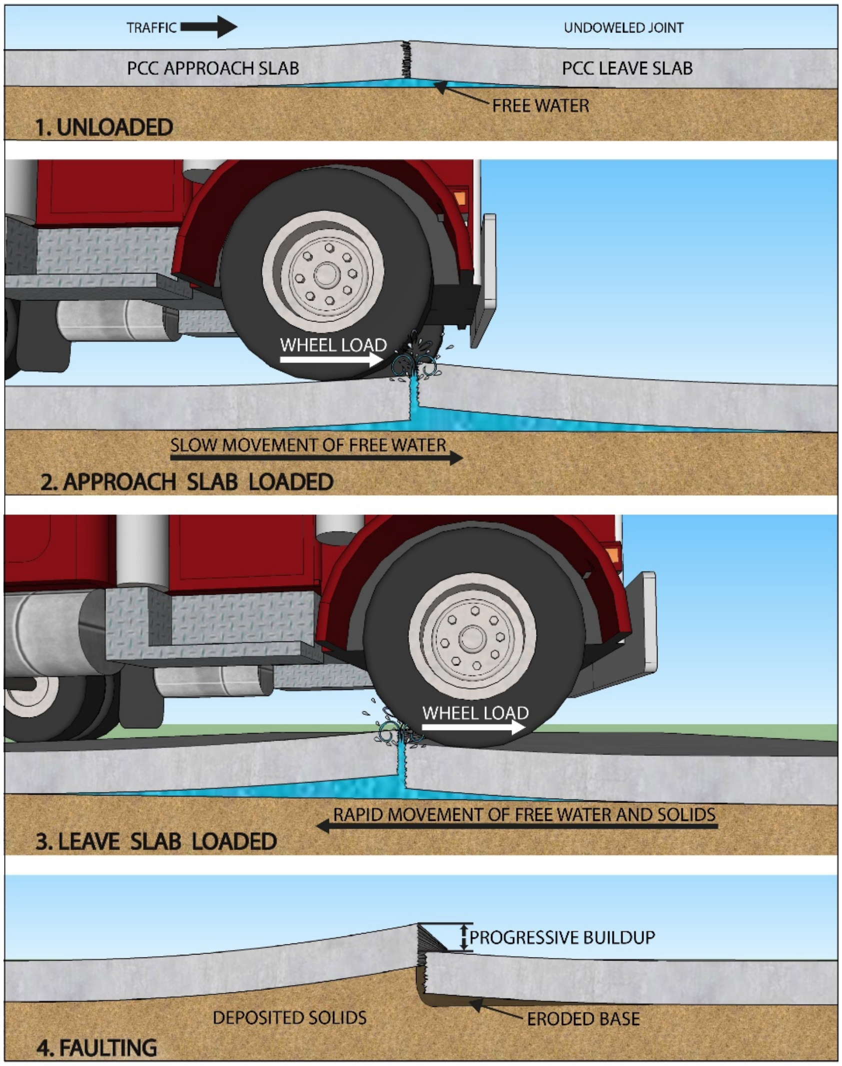
Prevention

Cracking can be prevented by providing adequate subgrade compaction, ensuring uniform subgrade, utilizing proper mix design, adhering to hot and cold paving recommendations, and providing proper jointing, among many others. Pavement's foundation (subgrade and/or subbase) is one of the most critical design factors in achieving excellent performance for any type of pavement (Zollinger and Bakhsh 2014). For additional information, cracking causes and prevention are thoroughly discussed in several chapters of this manual.

Pumping

Pumping is the expulsion of subgrade soil, base, and water due to traffic through a pavement joint (See Figure 9.6), crack or pavement/shoulder edge resulting in lack of load transfer from loss of subgrade support and thus faulting. Once the load transfer is lost, continued traffic loadings pump subgrade soils in the open joint/crack. Typically, this mechanism pumps material from the leave side of the joint/crack and accumulates material in the approach side or drives material completely out of joints and cracks. The reverse of this phenomena may occur in increasing vertical curves. As the slab rebounds, a void is left under the joint/crack and pavement faulting develops from lack of uniform support.

Figure 9.6 Pumping is the expulsion of subgrade soil, base, and water due to traffic loading

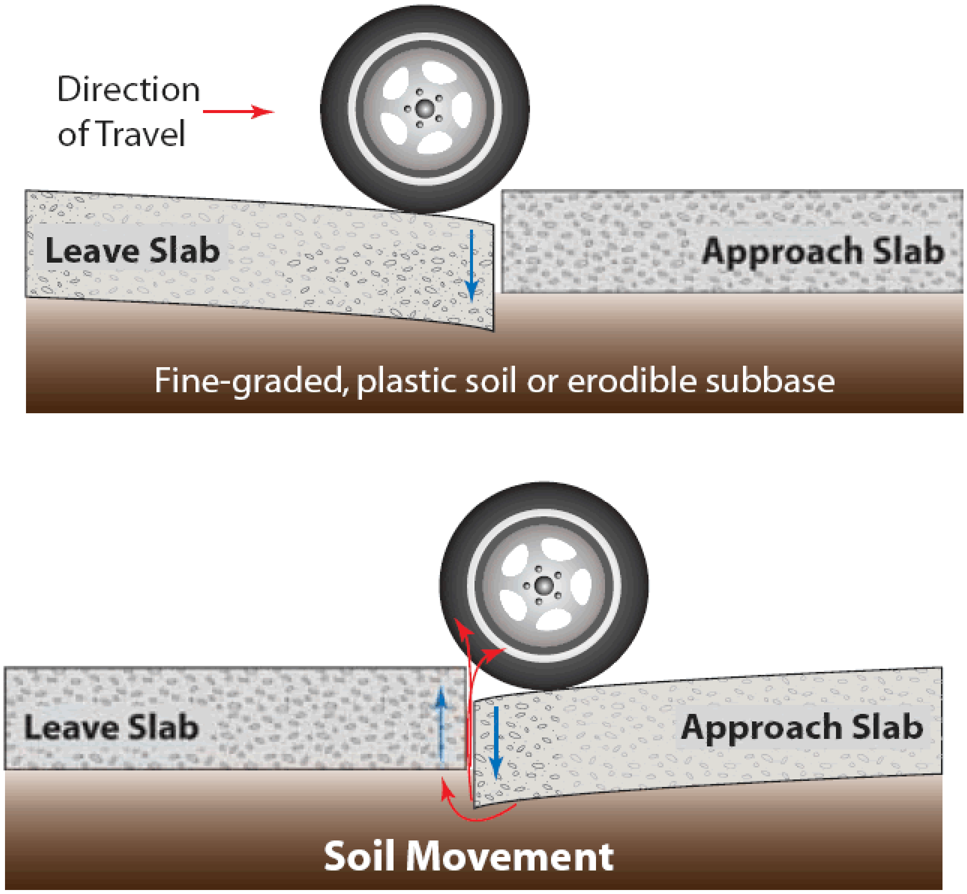


Minnesota DOT

Cause

Poorly drained plastic subgrade soils provide an environment to exacerbate conditions that promote pumping of subgrade soils out through pavement joints and cracks. A good pavement substructure needs uniform support with adequate drainage to minimize pumping of subgrade fines up into the pavement base/subbase. State-of-the-art design and construction procedures of pavement foundations are important aspects in assuring long-term pavement performance. It is important that the subgrade fines do not extensively contaminate the base/subbase of the pavement (see Figure 9.7).

Figure 9.7 Pavement movement from eroded subbases

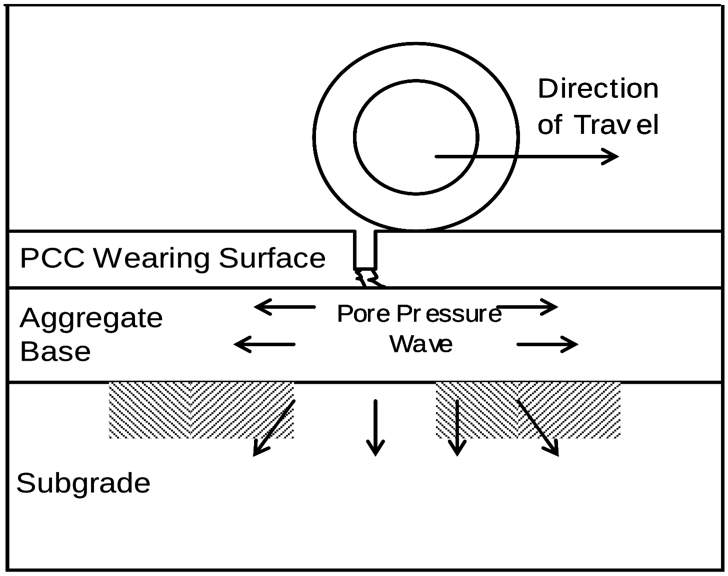


Snyder & Associates, Inc.

In the presence of water, the mitigation of fines from a subgrade soil into the overlying granular subbase of a highway depends mainly on the pore pressure developed in the interface between the subgrade and subbase (see Figure 9.8).

Sources of free water in the pavement include water infiltrating through cracks and transverse joints in the pavement, longitudinal pavement/shoulder joints, water from ditches and medians, and a high groundwater table.

Figure 9.8 Pore pressure in the aggregate base and subgrade



Snyder & Associates, Inc.

According to the *Guide for Design of Jointed Concrete Pavements for Streets and Local Roads* (ACI 2002), foundation-related factors that can contribute to pavement distress include those listed below.

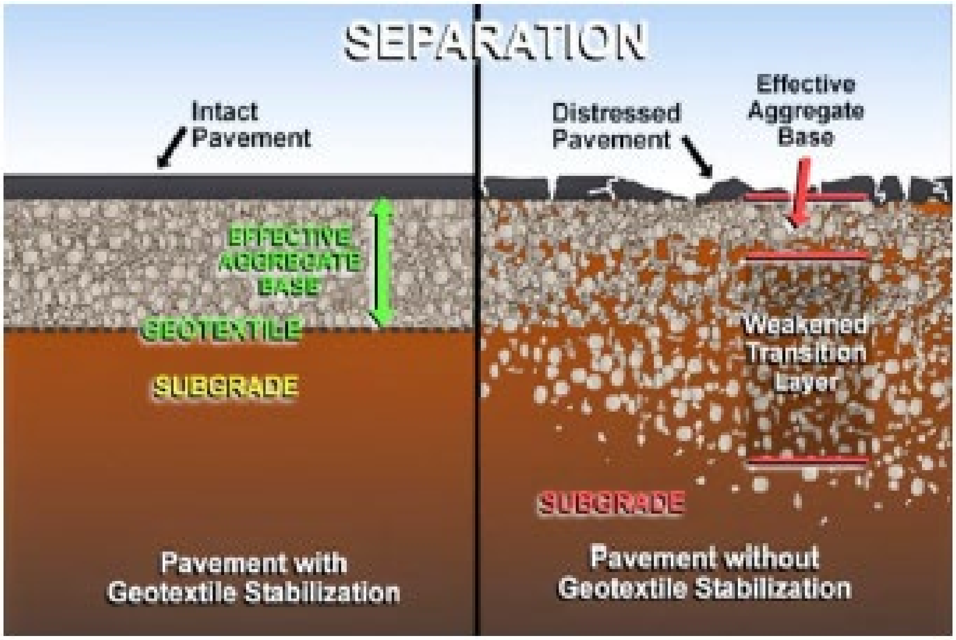
- Nonuniformity of support caused by differences in subgrade soil strength or moisture
- Nonuniform compaction
- Poor drainage properties of the subbase or subgrade can enhance the potential for erosion under the action of slab pumping, lead to loss of support, and ultimately faulting at the joints
- Nonuniform frost heave
- Excessive swelling of expansive subgrade materials

Prevention

Consider the following to minimize pumping risk.

Provide a well-designed drainable subgrade and subbase. When a subdrain system is utilized, provide proper maintenance, particularly at the outlet. If an unstabilized granular subbase is used, provide a minimum thickness of 6 inches. Construct a drainable base stable enough (proper amount of fines) to allow for construction traffic during the placement of the concrete pavement. As mentioned above in plastic subgrade soils, migration of the subgrade soils into the unstabilized granular subbase is a real possibility and can reduce pavement life (see Figure 9.9). Under this condition, increase the thickness of unstabilized granular subbase to account for 3 to 4 inches of loss through migrations of fines over time (Gross and Harrington 2014). Perhaps a more cost-effective alternative is to provide a geosynthetic separation layer between the subgrade and a thinner (5-inch thick minimum) unstabilized granular subbase.

Figure 9.9 Prevention of fines into granular unstabilized subbase



Frank Pace, Propex

Using mechanical load transfer devices (dowel bars) in the transverse joints when pavement traffic counts warrant a pavement thickness greater than 7 inches. Utilize tie bars for longitudinal joints that help maintain the aggregate interlock between panels. When pumping is a future concern for longitudinal joints, consider keyed longitudinal joints since the key will help maintain the load transfer, particularly in left and right turn lanes with heavy truck traffic.

Sealing joints for non-granular or non-stabilized subbase roadways is important to help prevent faulting.

Table 9.3 provides a summary of causes and prevention of faulting distress of concrete joints and cracks in JPCP and JRCP.

Table 9.3 Summary of causes and prevention of faulting distress of concrete joints and cracks in JPCP and JRCP

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Lack of Load Transfer	Failure of aggregate interlock Coarse aggregate degradation in concrete mix Failure of mechanical load transfer mechanisms Loss of support Excessive use of expansion materials	Proper mechanical load transfer design High-quality aggregates	Specify well- graded aggregates in concrete mixes that meet durability requirements	When specified, seal joints properly Provide adequate homogeneous subgrade support Provide quality compaction of pavement substructure	Periodic inspection of pavement Rehab before faulting reaches 1/2 in. Provide soil stabilization Retrofit load transfer
Cracking	Long panel lengths Misaligned dowel bars High steel reinforcement leading to corrosion Nonhomogeneous subgrade support Late sawing Change or loss of support	Adequate consolidation of subgrade Require low deformation properties of subbase Do not exceed 15-ft panel lengths Specify coating of reinforcement Proper reinforcing steel requirements	Provide corrosion-resistant load transfer and reinforcing materials	Make sure dowel bar baskets are properly anchored and reinforcing is properly placed	Provide partial- and full-depth repairs as needed

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Pumping	Compression shear from deflection of slab, heavy truck traffic, lack of load transfer, or poor subgrade support Saturation of joint/crack Pumping of subgrade fines Marginal aggregate soundness Saturation of joint or crack due to poorly sealed or unsealed joints on poorly draining grades Lack of edge drains	Proper drainage of subbase Seal joints Install edge drains Provide adequate mechanical load transfer Seal joints for low-speed traffic roadways and those having slow-draining substructures	Specify well-graded aggregate that meets durability requirements Provide adequate base thickness Use high-quality cold pour, hot pour, or preformed sealers	When specified, seal joints properly Provide adequate homogeneous subgrade support Provide quality compaction of pavement substructure Make sure joints are clean and dry when applying sealants	Rehab before faulting reaches 1/2 in. Proper maintenance of sealants in joints Periodic inspection and maintenance of subdrain outlets Early detection of spalling allowing the use of partial-depth repairs Fill voids under pavement to stabilize joints and cracks Install edge drains

6. Treatment and Repairs

The condition of the pavement structure should always be considered in determinations when recommending repairs. Preliminary investigations must identify the cause of distress. Expected pavement life of 5 years versus 20 years will have a dramatic effect on restorative strategies. Pavements with little remaining structural life are not good candidates for most of the restorative techniques described herein.

Faulting affects rideability and reduces service life of concrete pavement, therefore, early corrective action is imperative. Faulting becomes noticeable to the user at a depth of 1/8 inch or greater and needs rehabilitation before the faulting reaches 3/8 of an inch.

The use of modeling to predict faulting can be accomplished by utilizing AASHTO's *Guide for the Design of Pavement Structures* (1998) Section 3.3 Rigid Pavement Joint Design (pages 138–144) as a function of joint spacing, load transfer, lane width, subgrade and base type, climate, and traffic. Predictions can be made of the mean joint faulting over the design using models established for both doweled and non-doweled jointed concrete pavements.

Stabilization/Jacking – Void Detection

One cause of distress in concrete pavements is due to the lack of support under the pavement especially at the joints and cracks. Often these voids are no deeper than 0.125 inches. The soil stabilization process includes pumping a cement-grout or polyurethane material into the voids to re-establish pavement support. Slab stabilization restores slab support by decreasing deflections under a load. Be careful that the stabilization process does not create differential support.

Pavement investigation includes visual inspection, slab deflection measurement, and ground penetrating radar.

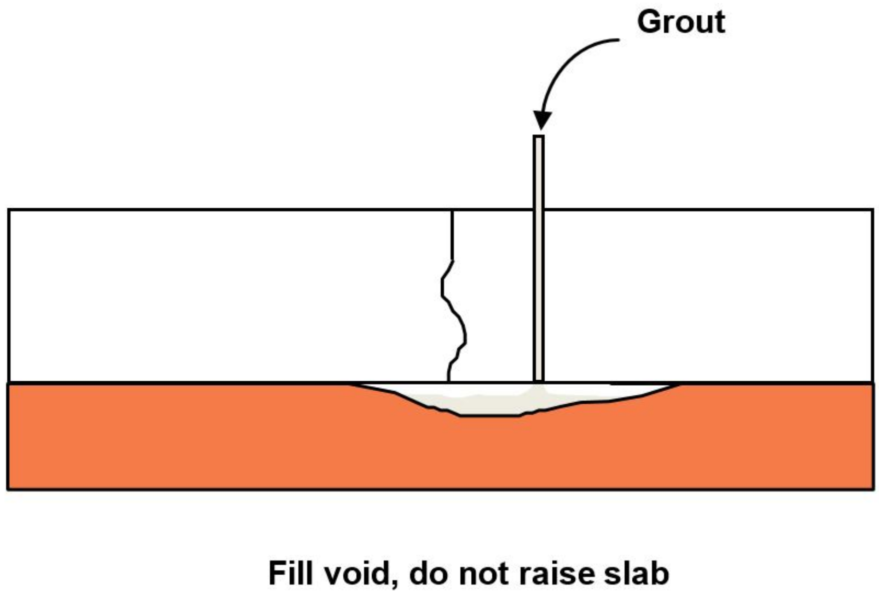
According to ACPA's publication *Slab Stabilization Guidelines for Concrete Pavements*, slab stabilization involves the following.

1. Determining optimal time to stabilize
2. Accurately detecting voids
3. Selecting acceptable stabilization materials
4. Correctly estimating material quantities
5. Using appropriate construction practices

Lack of subgrade support is noted by the pumping of subgrade and base material through joints and cracks as detected by visual inspection. This requires a more thorough analysis by FWD measurements and perhaps ground penetrating radar. This data is used to determine the optimal time to stabilize. Cement grout or polyurethane material is pumped through a number of injection holes drilled into the concrete. The adjacent holes are monitored to determine when the voids are filled. Preliminary testing is performed to estimate material quantities.

In badly faulted sections, additional pumping pressure may be necessary to jack the slab on the low side of the joint/crack and carefully lift the panel so that the elevation of both panels coincide (see Figure 9.10). Care must be used to avoid creation of nonhomogeneous support resulting in cracked panels.

Figure 9.10 Pumping grout



Doug Schwartz, Gateway Engineering and Training LLC

These severely faulted panels are usually localized and corrective action may require substantial subgrade work or complete panel replacement.

Since slab stabilization is somewhat of an art, it is recommended that it is a requirement to furnish a number of references indicating experience with this technology before implementation.

Concrete Pavement Repair

After subgrade support issues are addressed, the process of concrete pavement rehabilitation (CPR) of the distressed concrete pavement occurs. This process involves partial- or full-depth repairs, and complete concrete panel replacement as needed.

Edge Drains

Improving drainage will enhance the expected life of the pavement and reduce faulting risks. Moisture conditions should be investigated in the base/subgrade that may have accelerated the pumping of fines which initiated the pavement faulting. If the pavement and substructure continues to remain saturated, additional corrective action will be needed. This may involve the installation of edge drains to provide a route for subgrade water to escape, thereby increasing pavement support. It is extremely important that drainage outlets are monitored and debris is cleaned out to allow free water flow out of the pavement substructure.

Load Transfer Restoration (LTR)

After the distressed concrete pavement is repaired and the base support is re-established or verified, lost load transfer needs to be restored. Load transfer restoration involves placing dowel bars or other mechanical devices across joints and cracks. See Concrete Pavement Rehabilitation - Guide for Load Transfer Restoration (FHWA 1997) for further information on load transfer restoration. Load transfer restoration construction steps involve the following (see Figure 9.11).

1. Cutting slots in pavement (Figure 9.12)
2. Preparing slots
3. Placing dowel bars
4. Backfilling slots
5. Opening to traffic

Figure 9.11 Load transfer restoration

Minnesota DOT

Figure 9.12 Cut slots in pavement

Minnesota DOT

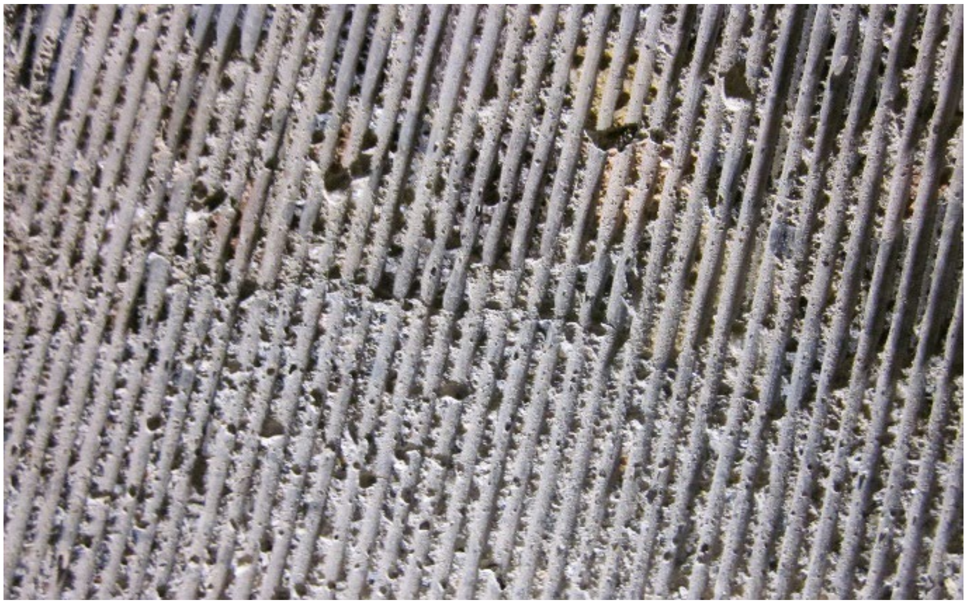
One common strategy for improving load transfer of transverse joints consists of placing dowels in the outside wheel path in both directions. Typically, three slots at 12 inches apart are cut mid-depth of the concrete pavement in each wheel path to allow placing of 1-1/4 inch to 1-1/2 inch diameter-round epoxy coated dowel bars 14 inches long with end caps to allow for expansion and contraction of the joint or crack. The concrete fins in the saw cuts are removed and the slots are prepared for installation of the dowel bars. The dowel bars are set on chairs to ensure horizontal and vertical alignment at mid-panel depth. Cracks and joints are caulked to prevent the intrusion of backfill material. The backfill material is placed and finished. Material is a proprietary to the concrete mixture, typically used for partial-depth concrete repairs. After restoration measures take place, the roadway can be open to traffic in approximately 2 to 6 hours.

Very high-traffic roadways may require additional and larger diameter dowels each spaced at 300 mm across the full-panel width.

Diamond Grinding

The final operation before re-sealing joints is to diamond grind the concrete surface to remove the faulting (see Figure 9.13) and restore the ride while improving the skid resistance of the concrete pavement, which also provides noise abatement. With a proper grooving depth, the procedure leaves a pavement surface texture that is “whisper quiet.”

Figure 9.13 Diamond grinding



Larry Schofield, American Concrete Paving Association

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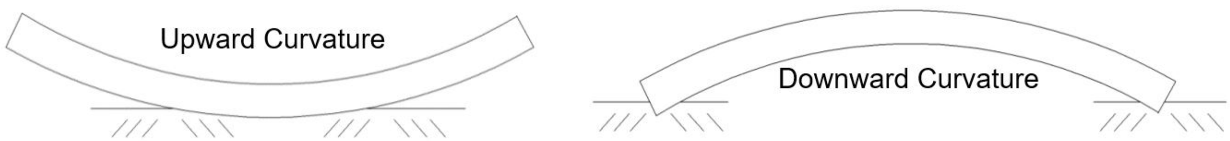
CHAPTER 10. JOINT CURLING AND WARPING

1. Description

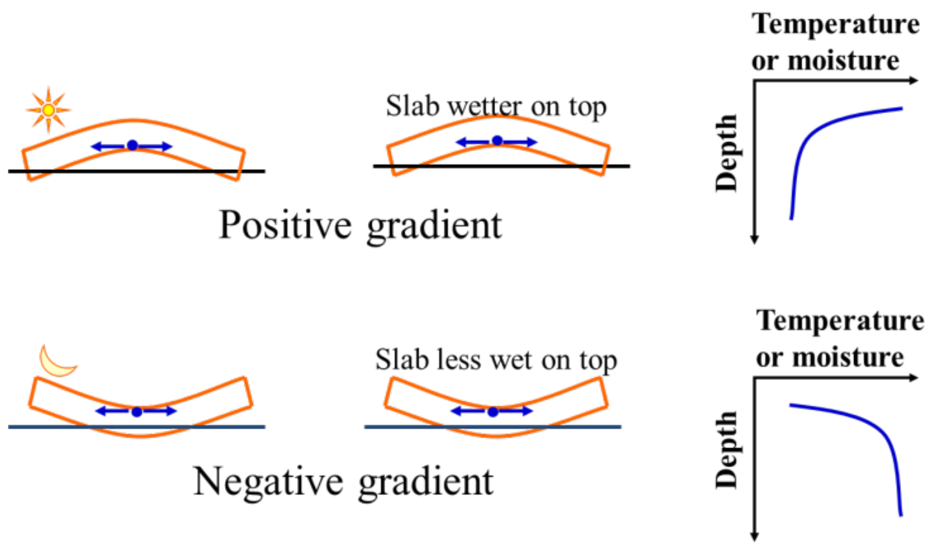
Concrete slabs placed on grade undergo nonuniform volumetric changes due to temperature and moisture gradients. A gradient is defined as the variation that occurs in temperature and/or moisture from the bottom of the concrete slab to the top. With regards to the temperature gradient, this changes throughout the day. The slab is normally colder on the top than the bottom from late at night through mid-morning, resulting in a negative temperature gradient. Under these conditions, the slab will have a tendency to undergo upward curling caused by the lower surface temperature. The term curling is reserved for curvature that develops in the concrete slab due to a temperature gradient. As the slab warms throughout the course of the day, the temperature at the top becomes greater than that at the bottom, developing a positive temperature gradient, which has a tendency to develop downward curvature as the concrete at the surface expands (Weiss 2015). Exaggerated upward and downward curvatures are illustrated in Figure 10.1.

Figure 10.1 Illustration of upward and downward slab curvature



Similarly, moisture gradients form in concrete pavements. In contrast to temperature gradients that fluctuate daily, the long-term moisture gradients are almost always negative, with the top of the slab being drier than the bottom. This results in upward warping from the shrinkage that develops in the slab surface as it dries. The term warping is reserved for curvature that develops in the concrete slab due to a moisture gradient. Figure 10.2 illustrates the impact of temperature and moisture gradients on slab curvature; the arrows illustrate the location where stress is generated due to the slab's self-weight.

Figure 10.2 Curvature due to temperature curling and moisture warping



Mack 2009

Curling and warping are rarely observed but are measurable through the use of slab profiling equipment (such as a rod and level survey or a Face Dipstick®), or through detailed analysis of profile data collected by inertial profiling devices.

Figure 10.3a shows a picture of upward slab curvature along the outside edge of the raised transverse joints. The ride quality on this pavement was severely affected by the upward curvature, to the point where the roadway had to be milled. The photo was taken before the outside lane milling was completed. Note the dowel bar retrofit slots that are being installed to try to help with the upward curvature.

Figure 10.3 Upward and downward slab curvature

a. Upward slab curvature visible at the raised transverse joints along the outside edge of the mainline due to the milling of the mainline

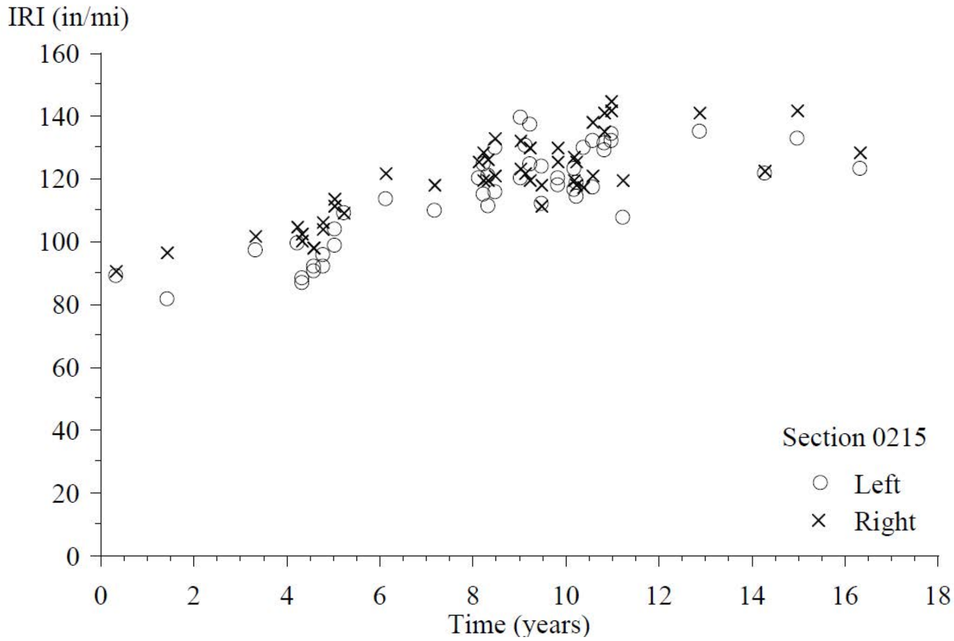
Daniel Frentress, Frentress Enterprises, LLC.

b. Downward slab curvature visible at a transverse joint under straight edge
Todd LaTorella, Missouri/Kansas Chapter, American Concrete Pavement Association

It has long been recognized that jointed concrete pavements can experience upward and downward curvature on a daily basis due to changing temperature gradients (curling), and over time, develop upward curvature due to moisture gradients (warping). Furthermore, recent research has demonstrated that both temperature curling and moisture warping are known to affect ride quality but in some cases warping has been identified as the dominant factor (Asbahan and Vandenbossche 2011, Karamihas and Senn 2012). Figure 10.4 shows the International Roughness Index (IRI) progression over nearly 17 years in both wheel paths of Arizona SPS-2 site Section 04 0215 in which detailed IRI data was collected seasonally, and on certain days, both in the morning and afternoon of the same day (Karamihas and Senn 2012). This pavement had no visual distress and no faulting. Figure 10.5 is from the same data set but is for the left wheel path only. In this data, the influence of curvature was mathematically removed allowing the effects of diurnal temperature on curvature to be examined. There are a number of striking conclusions that this data tells, including the following.

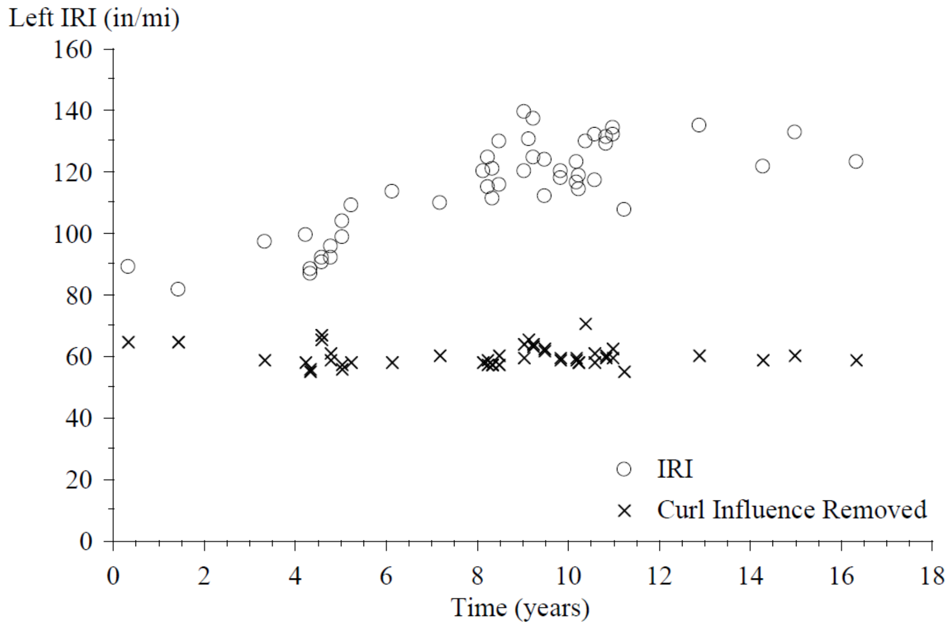
- All of the roughness that occurred after construction is due to the development of upward curvature. When the influence of curvature is removed, the IRI is steady at approximately 60 inches per mile for nearly 17 years.
- Roughness continued to increase for approximately 9 years due solely to increasing curvature, resulting in the IRI increasing from approximately 90 inches per mile for the 3 months following construction to 130 inch per mile at year 16.
- Curvature initially decreased within the first 2 years after construction and then increased thereafter until the pavement was approximately 9 years old. This trend was observed in other AZ SPS-2 sections, as well as in other projects (Karamihas and Senn 2012).

Figure 10.4 IRI data collected on Arizona SPS-2 Site 04 0215



Karamihas and Senn 2012

Figure 10.5 IRI data collected in the left wheel path on Arizona SPS-2 Site 04 0215



Karamihas and Senn 2012

On some days, data was collected in the morning and the afternoon. The effect of the changing temperature gradient on curvature is clearly seen in the data from year 8 to year 10, demonstrating that temperature curling slightly reduces the IRI when downward curling is countering upward warping, or slightly increases the IRI when upward curling is adding to upward warping. Daily variations in IRI due to temperature curling are on the order of 10 to 20 inch per mile whereas the IRI change due to warping is approximately 65 inches per mile.

At the time of the first measurement (approximately 3 months after construction), the total IRI was 90 inches per mile. Once the influence of curvature was removed, the IRI was 64 inches per mile. This illustrates how quickly curvature can develop, and the negative impact that curvature can have on the acceptance of new jointed concrete pavements when using the IRI.

2. Severity

Curling and warping are not pavement distresses; therefore, they do not have assigned severity levels. Curling and warping may not even be visible to the naked eye (although it is detectable through the use of profile measuring equipment), yet the slab curvature negatively impacts the pavement ride quality, increasing roughness as assessed through such indices as IRI. All jointed concrete pavements curl and warp to some degree, although reinforced concrete pavement is less susceptible to curling and warping as the slab sizes (between the cracks) are so short that it has little impact on ride quality. See Table 10.1.

Excessive amounts of early-age curling and warping are of concern, as they may impact the acceptance of the pavement by the owner. In the long term, it has been documented that the IRI of jointed concrete pavements may increase significantly in the absence of any observable distress (e.g., faulting, cracking, spalling) as a result of increasing upward curvature alone (Karamihas and Senn 2012). Furthermore, there is direct linkage between slab curvature and structural support, with upward curvature amplifying the generation of tensile stress in the surface of the concrete pavement, possibly leading to top-down cracking in concrete slabs (Asbahan and Vandenbossche 2011) or corner breaks when the slab has curled off of the underlying support. And rarely, excessive curvature has been linked to delamination at the transverse joints due to the development of high levels of stress caused by the restraint of curvature by the dowels, as well as the increased dynamic loading imparted by trucks that were harmonically affected by the curvature (Figure 10.6). In this image, the cracks perpendicular to the joint are above dowels. Coring verified that the delamination emanated at the depth of the dowels, scalloping outward from the joint as if it was a large spall.

Figure 10.6 Delamination cracking from excessive upward curvature

Van Dam

Table 10.1 Summary of curling and warping

Distress	Description	Severity	Measurement
Curling	Curvature that develops in the concrete slab due to temperature gradient	All jointed concrete pavements curl and warp to some degree Excessive amounts of early-age curling and warping are of concern	Slab curvature due to curling or warping can accurately be measured through detailed profiling of the slab using a rod and level survey or specialized equipment such as the Face Dipstick® Detailed analysis of profile data collected from an inertial profilometer can also measure slab curvature
Warping	Curvature that develops in the concrete slab due to a moisture gradient	All jointed concrete pavements curl and warp to some degree Excessive amounts of early-age curling and warping are of concern	Slab curvature due to curling or warping can accurately be measured through detailed profiling of the slab using a rod and level survey or specialized equipment such as the Face Dipstick® Detailed analysis of profile data collected from an inertial profilometer can also measure slab curvature

Measurement

Since curling and warping are not distresses, there is no way to accurately measure the degree of curvature using visual means alone. At night under oblique lighting, curvature is readily visible but the level of curvature can only be subjectively assigned. It is well established that the IRI is sensitive to slab curvature. Thus, an initial way to evaluate slab curvature would be to use IRI data as a trigger; however, this only works as long as no other roughness-inducing distresses are present. Unfortunately the influence of curvature cannot easily be separated from other sources of roughness; the method used by Karamihas and Senn (2012) required intensive data analysis that has not yet been automated. In time, algorithms may become available to remove the influence of curvature from profile data collected by an inertial profilometer rather easily, at which point curvature may be measured directly from the collected profile data.

Until that happens, the only alternative for measuring curvature is to do it directly. This can be done with surveying equipment (rod and level) or by using a Dipstick® profiler, which is commonly used to measure flatness and levelness on concrete floors. Although criteria have been established for flatness and levelness for concrete floor applications, no such criteria have been accepted for widespread use to establish acceptance levels of curvature for jointed concrete pavements based on Dipstick® measurements.

3. Testing

Following the visual site evaluation, it may be necessary to conduct certain evaluation tests to try to measure the degree of curvature and to try and isolate whether curvature is due to curling, warping, or a combination of the two. Refer to Chapter 19 for more detailed testing information.

Field Tests

Using surveying equipment (rod and level) or by using a Dipstick® profiler, the measurement of curling and warping can be accomplished. To determine if the curvature is related to thermal gradients (i.e., curling), multiple profile measurements can be done on a series of slabs over the course of a few days. Daytime and nighttime measurements will be needed, as will the installation of thermal probes at near surface, mid-depth, and near the slab bottom to assess the temperature gradient. Slab curvature can then be correlated to the thermal gradient to determine how much of it is due to temperature curling.

Coring: Core samples can be obtained for laboratory testing of the coefficient of thermal expansion and to assess concrete HCP volume.

Laboratory Tests

Temperature curling is related to the concrete coefficient of thermal expansion (CTE), which can be determined from core specimens obtained in the field in accordance with AASHTO T 336. There is no laboratory test that can be used to directly assess slab warping potential due to drying shrinkage on field extracted concrete as the shrinkage has already taken place. Drying shrinkage of a mixture can be assessed during the mixture design process using ASTM C157. HCP volume is known to be related to drying shrinkage and thus to slab warping. HCP volume can be measure using ASTM C457, Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete in combination with ASTM C856, Standard Practice for Petrographic Examination of Hardened Concrete.

4. Identification of Causes

Curling and warping are caused by temperature and moisture gradients, respectively, that result in differential volume change in the top of the slab compared to the bottom. The cause of excessive curvature is primarily materials-related, although elements of design and construction can play a role (see Table 10.2).

Table 10.2 Physical and material/chemical causes of joint warping and curling

Distress	Category	Description
Curling (Physical)	Temperature Gradient	The temperature gradient changes constantly throughout the day, resulting in curling.
Curling (Material/Chemical)	CTE	The CTE defines how a material changes in length with a unit change in temperature. Amount of aggregate directly influences the CTE. Higher CTEs produce increased potential for curling.

Distress	Category	Description
Warping (Material/Chemical)	Drying Shrinkage	Drying shrinkage characteristics of the concrete directly impact warping; mix water is a critical factor.
Warping (Physical)	Subsurface Drainage	If water cannot drain from beneath a pavement, warping will be exacerbated due to an increased moisture gradient.

Concrete Material Factors Contributing to Curvature

Factors that affect volume change in concrete as a result of changes in temperature and moisture will impact the development of curvature in the slab. As each of these are distinctly different, they will be addressed separately.

Concrete Material Factors Contributing to Curling

Curling is a result of a temperature gradient in the concrete slab. This gradient is constantly changing as the pavement surface is warmed in the heat of the day, and then cools at night. One factor that can influence this is the color of the concrete, as it is closely related to the amount of solar energy absorbed into the concrete. The surface of darker concrete, particularly concrete that has been colored, will get hotter under direct sunlight and thus will have a higher positive temperature gradient than a lighter concrete, all other things equal. But overall this effect is thought to be minor.

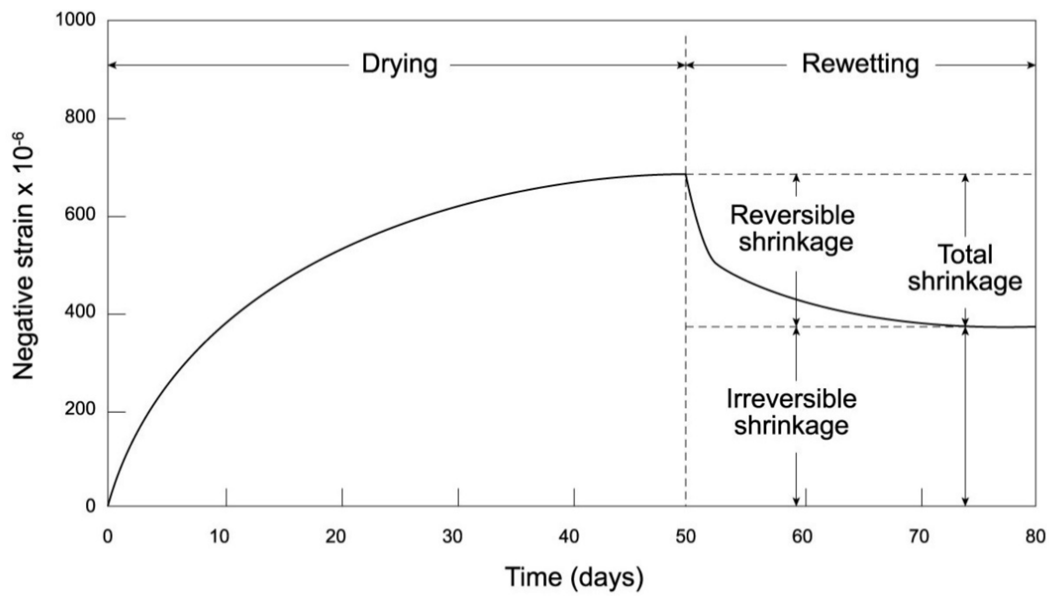
The material factor that has the greatest impact on the development of temperature curling is the concrete's coefficient of thermal expansion (CTE) measured in accordance with AASHTO T 336. The CTE defines how a material changes in length with a unit change in temperature. It is reported as the change in length of a specimen over the total specimen length per degree change in Fahrenheit (°F) or Celsius (°C). For concrete, CTE typically ranges from 4 to 7.5 x 10^{-6} inch/inch/°F, with lower values indicating less length change for the concrete for a given change in temperature. As the volume of concrete is predominately aggregate, the aggregate (particularly coarse aggregate) has a large influence on CTE. Pure limestone aggregates produce concrete with the lowest CTE values, whereas concrete produced with siliceous aggregates such as quartzite have the highest CTE values (FHWA 2016). All variables equal, concrete having a higher CTE value will have an increased potential for curling. But it is recognized that high-quality, long-lasting concrete pavements are routinely produced with aggregates having a high CTE value as long as jointing and other design practices are such that the higher CTE value is accommodated.

Although daily cycles of temperature curling in the presence of traffic can amplify fatigue cracking of the slab, it must be remembered that temperature curling plays a relatively small role in the development of curvature that affects the ride quality of jointed concrete pavements (Asbahan and Vandenbossche 2011, Karamihas and Senn 2012); this is predominantly a result of moisture-induced warping.

Concrete Material Factors Contributing to Warping

Hardened concrete is primarily composed of aggregates held together by hydrated cement paste (HCP). The HCP is composed of minerals created through chemical reactions between the cementitious materials (cement and supplementary cementitious materials) and mix water. These minerals do not occupy all of the space that was once occupied by mix water, and thus there is considerable pore space, called capillary pores, that remains in the 0.01 to 5 micron size range (Mindess et al. 2003). Capillary pores have a large influence on concrete strength and permeability. They also have a large influence on the volumetric change that concrete undergoes with changes in moisture. As concrete dries, the pores begin to empty, starting with the largest pores first. In such small pores, as they transition from being full to partially filled with water, a menisci forms due to surface tension. The menisci in millions of pores pull on the pore sidewalls, drawing them in ever so slightly (McCracken et al. 2008). As drying continues, increasingly smaller pores become partially full. In smaller pores, the surface tension of the water increases, as does the stress pulling the pore walls inward. The net result is that the concrete shrinks as it dries, which creates the need for joints to accommodate the shrinkage. This is illustrated in Figure 10.7.

Figure 10.7 Plot showing shrinkage and partial recovery of concrete due to drying and re-wetting



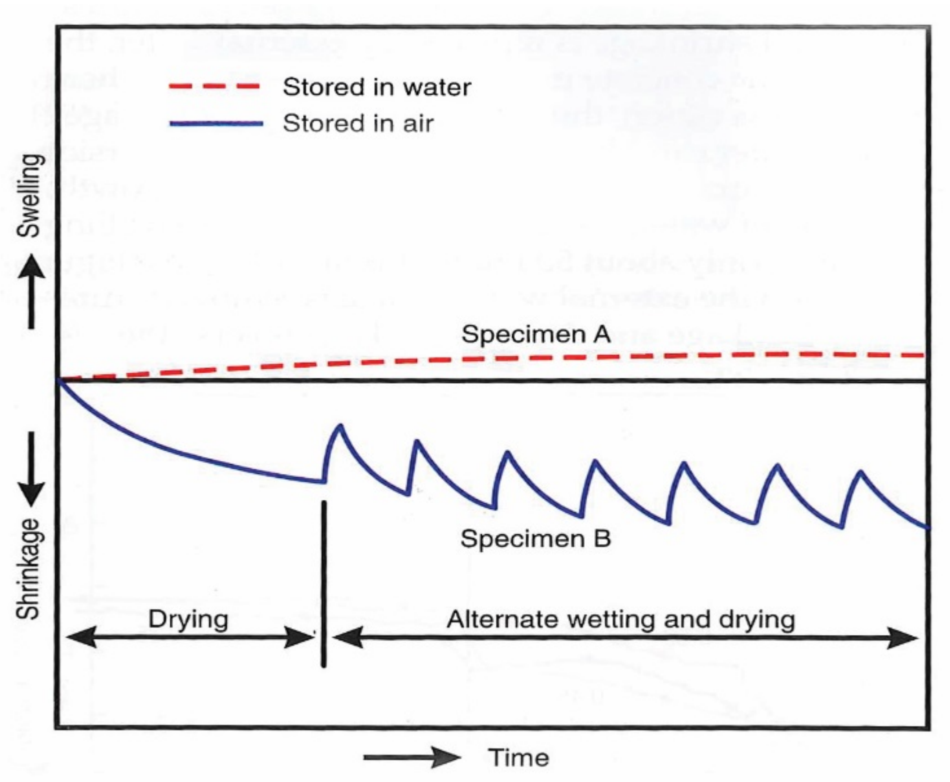
Recreated from Mindess et al. 2003, ©2003, 1996 Pearson Education, used with permission

As dry concrete is re-wetted, the opposite occurs. Empty or partially empty capillary pores take up water; the smallest pores first, until full and then the menisci disappear. The pore walls rebound but not to the same degree as where they started as permanent changes have occurred to the HCP microstructure. Thus, only a portion of the shrinkage is reversible as shown in Figure 10.7. The net result is that once concrete has dried, it will never return to the original volume it occupied when placed as a result of re-saturation alone. Furthermore, as the concrete undergoes repeated cycles of wetting and drying, the shrinkage due to drying continues to increase as illustrated in Figure 10.8.

This is in contrast to a specimen that is continually soaked that actually shows slight expansion. Taking these things into account, it can be seen how concrete slabs that undergo cycles of wetting and drying at the surface but remain largely saturated on the bottom will develop upward curvature that will continue to increase for years to come (Asbahan and Vandenbossche 2011, Karamihas and Senn 2012).

Thus, the key material property most directly influencing warping is the drying shrinkage characteristics of the concrete. This is largely controlled by the volume of HCP, which in turn, is controlled by the volume of cementitious materials and mix water present (AASHTO PP 84-17). Many believe that the amount of mix water added to the concrete is the critical factor (Kosmatka and Wilson 2016). Drying shrinkage can be measured using an unrestrained shrinkage test such as ASTM C157 but relationships between test results and degree of warping have not yet been established although efforts are underway to establish limits based on ASTM C157 to minimize slab warping (AASHTO PP 84-17).

Figure 10.8 Plot showing the effects of drying and wetting cycles on concrete shrinkage



Kosmatka and Wilson 2016, ©2016 Portland Cement Association, used with permission

Design Factors Contributing to Slab Curvature

From a design perspective, curvature can be minimized through restraint, including the use of dowels and tie bars at joints (Asbahan and Vandenbossche 2011). But the restraint of curling and warping, through the self-weight of the slab, bonding with the subbase, and/or the presence of dowels and tie bars at joints, results in the generation of stress that may lead to slab cracking. It is well established that using concrete with a lower CTE allows for the use of longer joint spacing and/or reduced slab thickness in mechanistic-empirical (ME) design such as the AASHTOWare Pavement ME Design software. Similarly, drying shrinkage is accommodated to some degree within Pavement ME through inputs such as concrete mixture paste volume and ASTM C157 results but the effects of slab curvature are not directly computed. Instead, the software evaluates the impact of the material properties on the development of stress and the development of cracking and changes in IRI.

In essence, if low CTE, low drying shrinkage concrete mixtures are chosen, the risk of developing unacceptable curvature is reduced. Designers can use this information in an ME design procedure to extend the joint spacing to meet the design requirements (e.g., distress, IRI) if concrete with low susceptibility to volumetric change is to be used. Alternatively, if the concrete mixture is highly susceptible to volumetric change, the designer can shorten the joint spacing and/or adjust other parameters to minimize the risk of curvature induced distress and roughness.

Another design element that may have an impact on warping is subsurface drainage. If water cannot drain from beneath a pavement, the concrete can remain at or near saturation, and will undergo little to no shrinkage (as shown in Figure 10.8, it may even experience slight expansion). This situation will exacerbate warping due to shrinkage occurring at the concrete surface by increasing the moisture gradient. The design of a drainable subbase and a good subsurface drainage system can reduce the risk of fully saturating the concrete at the bottom of the slab and reduce upward curvature due to slab warping.

Construction Factors Contributing to Curvature

Construction factors that play a role in the development of curvature include ambient conditions at the time of placement and curing.

Ambient conditions at time of placement, temperature, and rate of evaporation can play a role in contributing to curvature. When concrete pavements are placed in the heat of a hot summer day, the risk of cracking and early age curvature increases. For this reason, paving specifications routinely specify the temperature of the concrete mixture at time of delivery and contain hot weather concreting provisions to mitigate the negative impacts of high ambient temperature on newly placed concrete (Kosmatka and Wilson 2016). The major problem encountered when placing concrete during high temperatures is accelerated curing, which reduces the workability of the concrete. Relative humidity and wind also plays a role, along with temperature, in the rate of evaporation of bleed water from the concrete surface (ACI 2010b). It is noted that high rates of evaporation can be encountered even on relatively cool days if the relative humidity is low and the wind is high. If the rate of evaporation is high, the loss of moisture from the surface is more rapid than the rise of bleed water, which can result in plastic shrinkage cracking and poor curing of the surface.

Alternatively, cooler temperatures during initial curing can reduce ultimate shrinkage for a given mixture (Kosmatka and Wilson 2016). Although this may reduce warping for concrete pavements placed during cooler times of the year, it is recognized that this is not a viable strategy as it is largely out of the control of those specifying and constructing pavements.

It is believed that a temperature gradient can be “locked” into the concrete slab at time of set. This is not quite correct as the transition of concrete from plastic to solid is not instantaneous and during this time of transition, stress that would develop due to a temperature gradient is quickly dissipated through creep. Instead, “built-in curl” is more closely related to drying shrinkage in which moisture loss that occurs from the surface in the hours, days, and weeks following placement results in the formation of a moisture gradient that persists through the pavement’s life. This has been observed in field-placed concrete in which the temperature gradient at time of set is quite small in comparison to the effective built-in curl measured years later that developed primarily as a result of the loss of moisture from the surface (Asbahan and Vandenbossche 2011). The same trend is observed in field data, such as that of the AZ LTPP SPS-2 sites, where the curvature is always upward, and predominantly linked to continued drying.

Curing is addressed in more detail in the next section but it is important to emphasize here that rigorous adherence to recommended curing practices can help reduce the development of curvature. When using a white pigmented curing compound, it is essential that it be applied as recommended.

5. Evaluation

Slab curvature cannot be prevented, only minimized. To control slab curvature and minimize its impact on ride quality and distress formation, volumetric expansion and contraction of the concrete must be accommodated in the design, the concrete mixture cannot be susceptible to volumetric changes, and good curing practices must be followed.

Upward Slab Curvature (Design Elements)

Cause

If high CTE and drying shrinkage concrete mixtures are chosen, the risk of developing unacceptable curvature is increased significantly.

Additionally, poor subsurface drainage is another design element that increases slab curvature due to warping. If water cannot drain from beneath a pavement, the concrete will remain at or near saturation, and will undergo little to no shrinkage, maybe even expanding slightly. In contrast, concrete at the surface will undergo shrinkage as it undergoes cycles of drying and wetting, therefore exacerbating warping.

Prevention

The use of shorter slabs, dowelled joints, and bonding of the concrete slab to an underlying stabilized base are design elements that can help mitigate the magnitude of long-term upward curvature in jointed concrete pavements, reducing its impact on IRI. Note that all design elements must be considered together as a system to address anticipated traffic and site-specific environmental conditions as undesirable consequences can result if care is not taken (e.g., bonding to a stabilized base may increase the risk of cracking).

Alternatively, the elimination of transverse joints through the use of continuously reinforced concrete pavement (CRCP) is quite effective at minimizing upward curvature as the effective slab length is very short, being the distance between the naturally occurring transverse cracks (e.g., 3 to 8 feet). As more understanding is gained regarding the impact of long-term curvature on IRI, the calibration of the AASHTOWare Pavement ME Design models may need to be adjusted to fully account for changes in concrete material properties and design elements.

Furthermore, the subbase should be drainable and consideration should be given for the placement of a good subsurface drainage system that can prevent water from accumulating beneath the slab. A maintenance program must be implemented to ensure that the drainage system remains functional.

Slab Curvature (Mixture Elements)

Cause

Constantly changing temperature gradients in the concrete directly influence curling. Darker concrete (that has been colored) absorbs sunlight and gets hotter than lighter colored pavement, which induces curvature.

With regards to mixture constituents, it is a common industry belief that the most important factor affecting drying shrinkage is the amount of water added per unit volume of concrete (Kosmatka and Wilson 2016). The volume of water is related to the overall cementitious materials content of the concrete through the water-to-cementitious materials (w/cm) ratio. In combination, the volume of mix water and volume of cementitious materials define the total volume of HCP in the concrete.

In recent years, shrinkage-reducing admixtures (SRAs) have been developed that can significantly reduce drying shrinkage in concrete; however, they have not seen widespread use due to their high cost and unproven long-term effectiveness in pavement applications. Other properties of mixture constituents are also influential with regards to drying shrinkage, including the fineness of the cement, the type and volume of supplementary cementitious materials, the nature of the aggregates (including their absorptivity and degree of saturation at time of batching), and some admixtures (Kosmatka and Wilson 2016). Pre-wetted lightweight aggregates (PWLA), discussed further in the following sections, can also be effective in reducing early age drying shrinkage by providing an internal source of curing water.

Prevention

The major mixture constituent strategy to reduce temperature curling is usage of a coarse aggregate with a relatively low CTE, as this will produce a concrete with a lower CTE. The best method to assess this is to make concrete specimens representative of the mixture to be used on the project and determine the CTE in accordance with AASHTO T 336. The lower the CTE, the less temperature induced curling, all things equal.

For a given w/cm ratio, reducing the cementitious materials content through increasing aggregate volume will reduce the ultimate shrinkage of the concrete, not only because of the reduction in HCP but also because aggregates provide internal resistance to shrinkage. Typically, for paving grade concrete, a total cementitious content of less than 550 pounds per cubic yard is desirable for many reasons, including reducing shrinkage. This can be accomplished while maintaining a workable mixture by optimizing the aggregate gradation. Some state DOTs have reduced the cementitious content of their concrete paving mixtures below 500 pounds per cubic yard with great success. The new AASHTO PP 84-17, *Standard Practice for Developing Performance Engineered Concrete Pavement Mixtures*, has a prescriptive limit for the maximum HCP volume of 25 percent.

Water can also be reduced for a given cementitious materials content by reducing the w/cm but the w/cm of paving concrete is usually not reduced below 0.40 as this meets all durability requirements (ACI 2014). Concerns exist if the w/cm drops significantly below 0.40, autogenous shrinkage (due to chemical shrinkage and self-drying of the paste as water is consumed in hydration) becomes prominent (Kosmatka and Wilson 2016). On the other hand, some states including Minnesota have reported good performance with paving grade concrete typically having w/cm of 0.37. There is an added benefit gained from low permeability without suffering damage from autogenous shrinkage as the paste content is low and pavements are not as restrained as structures (such as bridge decks). Typically, a good range of w/cm for paving grade concrete is between 0.40 and 0.45, although higher values may be suitable for paving in areas not subjected to freeze-thaw or deicers.

As the importance of drying shrinkage increases, unrestrained shrinkage testing (e.g., ASTM C157) should be considered as part of the mixture design process for paving grade concrete. This helps to assess the drying shrinkage characteristics of proposed concrete mixtures as it pertains to the long-term development of upward curvature due to warping. AASHTO PP 84-17 sets a prescriptive limit of a maximum unrestrained shrinkage of 420 microstrain at 28 days as determined by ASTM C157.

Curing Practices to Mitigate Slab Curvature

Cause

Although it has always been thought that wet curing is the best way to cure pavements if it's practical, recent research has suggested that wet curing can actually increase warping in concrete slabs and thus might not be the best approach for curing slabs in dry environments (Hajibabaee and Ley 2015).

Poor application or tardiness in utilization of cure can result in slab curvature. Cure will form a membrane that significantly reduces evaporation while also making the concrete surface a highly reflective bright white, minimizing the absorption of solar energy and thus reducing heating at the concrete surface.

Prevention

Curing that slows or prevents mix water from evaporating from the pavement surface will help moderate the development of a moisture gradient and help control the rate of drying shrinkage and potentially the magnitude of ultimate drying shrinkage (Kosmatka and Wilson 2016). Curing compound should be applied as soon as the concrete surface has undergone final finishing and the bleed water has disappeared from the surface. The application must be uniform and thorough, and there must not be any thin areas or gaps in the coverage.

Proper use of effective membrane-forming curing compounds that hold free moisture in the concrete for long periods of time can help delay the onset of shrinkage, although long-term efficacy is still under investigation. Most curing compounds are white pigmented and thus will reduce solar gain onto the pavement surface to assist in reducing the positive temperature gradient that develops during the afternoons in the days immediately after placement. The use of PWLA has shown promise to improve curing and reduce ultimate drying shrinkage in concrete (Henkensiefken et al. 2009) but additional work is needed to determine the effectiveness of PWLA in reducing long-term upward curvature in concrete pavements.

Table 10.3 provides a summary of the mechanisms leading to curling and warping as well as design, materials, construction, and maintenance factors that can be used to prevent or mitigate it.

Table 10.3 Summary of causes and prevention of joint warping and curling

Distress	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Slab Curling	Temperature gradient in slab induces slab curvature and stress High CTE aggregates can contribute to higher levels of curvature and stress	Accommodate the potential effects of curling in design through proper joint spacing, load transfer, and base restraint Use of CRCP	When possible, use coarse aggregate with a low CTE	The temperature of the concrete should be maintained at or below specified limits during early curing White pigmented curing compound can assist in keeping surface cool The risk of early age cracking from curling can be reduced by not paving prior to a major temperature fluctuation (i.e., cold front)	There is no maintenance strategy that will specifically address curvature due to a temperature gradient Retrofit dowel bars at undowelled joints to provide restraint that reduces curling
Slab Warping	Moisture gradient in slab induces slab curvature and stress Arid climates, poor drainage, and concrete mixtures susceptible to shrinkage will contribute to higher levels of curvature and stress	Accommodate the potential effects of warping in design through proper joint spacing, load transfer, and base restraint Ensure that the pavement is free draining and does not trap water beneath the slab	Use concrete mixtures with lower potential for drying shrinkage (e.g., lower paste content) Internal curing may reduce drying shrinkage and warping	Minimize evaporation from the concrete surface at an early age; white pigmented curing compounds can help Paving during high temperatures can be a contributing factor; employ hot-weather concreting techniques	Ensure that drainage systems are maintained in pavements that are designed to drain Retrofit dowel bars at undowelled joints to provide restraint Diamond grinding to restore ride quality

6. Treatment and Repairs

Pavement ride quality and distress development should be monitored to determine the rate of IRI gain and whether the decrease in ride quality is attributable to distress. If little to no distress is observed, it is likely that slab curvature is responsible for the loss in ride quality. If this is the case, it is recommended that diamond grinding be conducted before the loss in IRI becomes too severe. Roughness due to slab curvature can be harmonic and exceptionally uncomfortable to the traveling public. Diamond grinding is a very cost-effective treatment. Further description of diamond grinding can be found in the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

In addition to the use of diamond grinding to restore ride quality, it may be beneficial to reduce the level of moisture that exists beneath the slab. If a subdrain system exists, it should be routinely inspected and cleaned as needed. Although the installation of a subdrain system might be tempting if it is determined that moisture beneath the slab is contributing to the development of curvature, the cost effectiveness of this approach should be determined. It is very difficult to sufficiently drain an existing concrete pavement that was not originally constructed with this in mind.

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CHAPTER 11. BLOWUPS

1. Description

A blowup is a result of localized upward movement or shattering of a slab along a transverse joint or crack (FHWA, 2014). Blowups often occur in the heat of the day as expansion results in a buildup of pressure that can be dramatically released as the pavement thrusts upwards and/or shatters. Although it is most commonly associated with jointed plain concrete pavement (JPCP), it has also been known to occur in continuously reinforced concrete pavement (CRCP). See Figure 11.1.

Figure 11.1 Common blowups of joints and cracks

a. Blowup in JPCP

Jeff Uhlmeyer, Washington State DOT

b. Blowup in JPCP
Jeff Uhlmeyer, Washington State DOT

c. Blowup in CRCP showing exposed steel
Miller and Bellinger 2014

2. Severity

There is no severity level assigned by the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) to blowups but they can be catastrophic and require immediate treatment to restore serviceability. In those cases where a blowup does not immediately impact traffic, the blowup can be considered to be of low severity but should be repaired as soon as possible. Blowups in JPCP and CRCP are of finite length and full-lane width. As such, the blowups per unit length (typically lane-mile) are simply counted. Table 11.1 provides a summary of the severity of blowups.

Table 11.1 Summary of severity of blowups

Distress	Description	Severity Levels	Measurement
Blowups	Result of localized upward movement or shattering of a slab along a transverse joint or crack	**Low**: Does not impact traffic **High**: Affects traffic and must be immediately repaired	Blowups per unit length (typically lane mile)

3. Testing

The coefficient of thermal expansion (CTE) defines how a material changes in length with a unit change in temperature. As the volume of concrete is predominately aggregate, the aggregate (particularly the coarse aggregate) has a large influence on CTE. All variables equal, concrete having a higher CTE value will have an increased potential for blowups. But it is recognized that high-quality, long-lasting concrete pavements are routinely produced with aggregates having a high CTE value as long as jointing and other design practices are such that the higher CTE value is accommodated. Additional information regarding the testing of CTE can be found in Chapter 19.

4. Identification of Causes

If a pavement has suffered blowups, the cause must be definitively identified. Blowups are caused by volumetric expansion in the concrete pavement that cannot be accommodated, resulting in stress that is relieved when a blowup occurs. The two primary causes are the presence of AAR or infiltration of incompressible materials into the joints. Other factors contributing to this include materials, pavement design, construction timing, and maintenance of the joints. Each is briefly described in Table 11.2 and in more detail under Evaluations.

Table 11.2 Physical and material/chemical causes of blowups

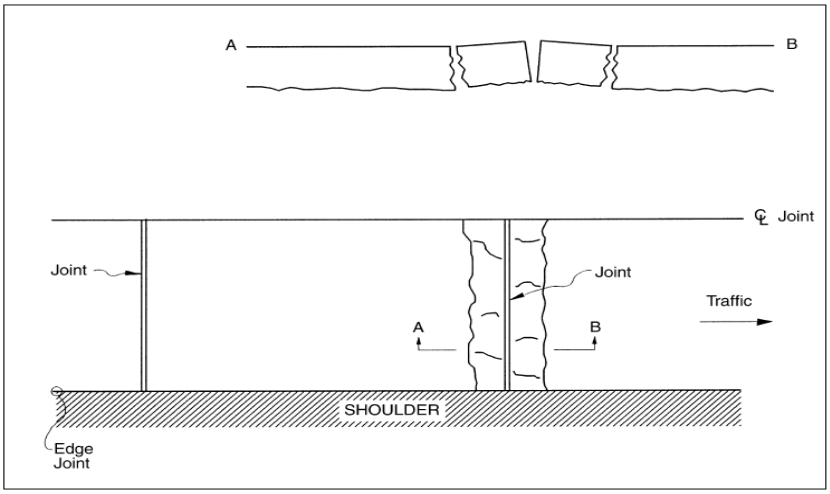
Distress	Category	Description
Blowups (Material/Chemical)	Evaporation	Evaporation of original water causes drying out, induces shrinkage, and leads to blowups.
Blowups (Material/Chemical)	Aggregate Deterioration	Pavement undergoes sizable expansion when affected by aggregate freeze-thaw or alkali reactions, such as alkali aggregate reaction (AAR), alkali carbonate reaction (ACR), and alkali silica reaction (ASR).
Blowups (Material/Chemical)	Coefficient of Thermal Expansion (CTE)	The CTE defines how a material changes in length with a unit change in temperature. Amount of aggregate directly influences the CTE. Higher CTEs produce increased potential for blowups.
Blowups (Physical)	Temperature Change	Blowups are often associated with heat waves because frequency increases as temperature increases.
Blowups (Physical)	Joint Length	Long jointed pavements are more susceptible to blowups.
Blowups (Physical)	Drainage	Poor pavement drainage (especially in cold weather states) increases the risk of blowups.
Blowups (Physical)	Time of Construction	Time of year of construction has been linked to occurrence of blowups.
Blowups (Physical)	Incompressible	The filling of transverse joints with incompressible materials is the most common cause of blowups, resulting in an increase of compressive stress and blowups.

5. Evaluation

Concrete Material Factors

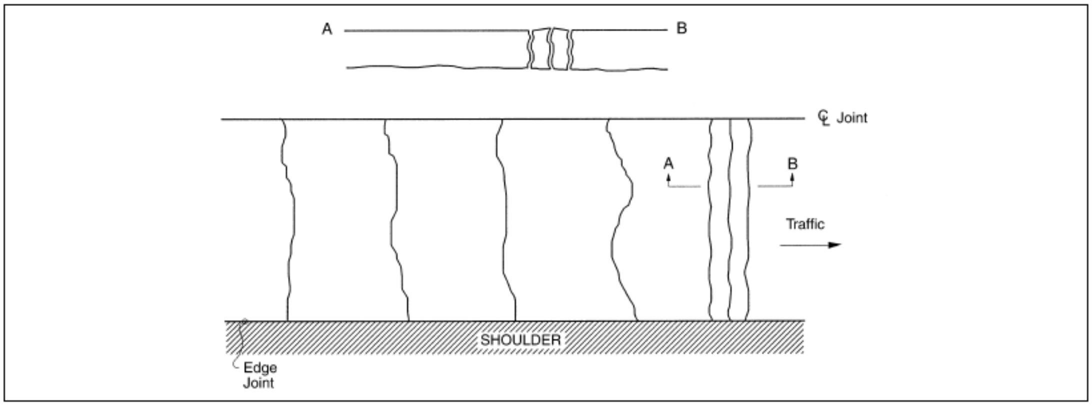
At a basic level, concrete is a mixture of aggregates, cementitious materials (i.e., cement, supplementary cementitious materials), water, and admixtures. The constituents are mixed, transported, placed, and finished in a plastic state, and as the cementitious materials chemically react with water, the concrete hardens. Over time, most of the original water is consumed in the chemical reactions or is lost to the environment, predominantly through evaporation. As the space previously filled with mix water (called capillary pore space) dries out, menisci form inducing shrinkage in concrete that is accommodated through the joints of JPCP or closely spaced cracks in CRCP. In most circumstances, the volume of the concrete is never greater than the day it is placed. There are exceptions, however, that sometimes lead to blowups. Material factors that can lead to expansion beyond the original concrete volume include moisture and/or temperature effects and expansive reactions. Each of these are shown below. Figure 11.2 shows a schematic of a blowup in jointed concrete pavement and Figure 11.3 shows a schematic of a blowup in CRCP.

Figure 11.2 Section of Blowup in JPCP



Miller and Bellinger 2014

Figure 11.3 Profile of blowup in CRCP



Miller and Bellinger 2014

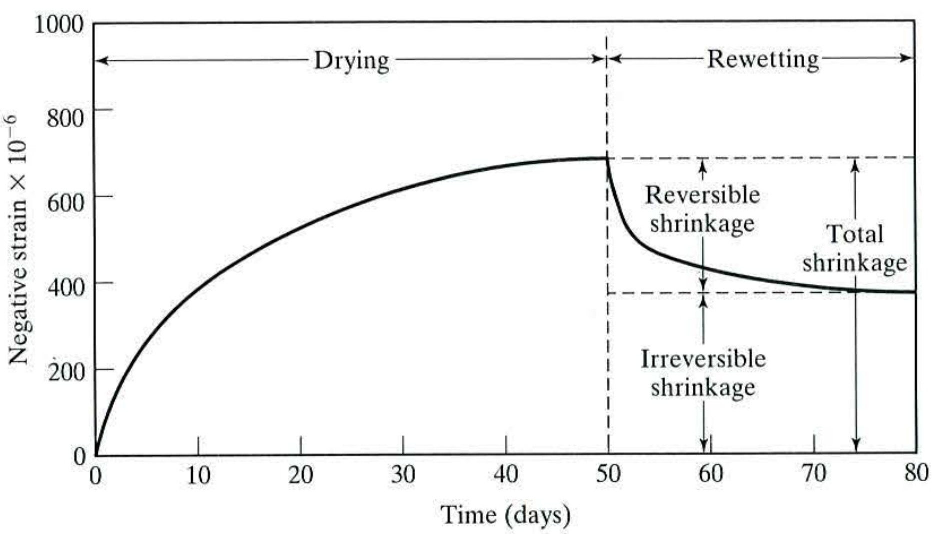
Volumetric Expansion Due to Moisture and/or Temperature Effects

Cause

In a broad sense, hardened concrete is primarily composed of aggregates held together by hydrated cement paste (HCP). The HCP is composed of minerals created through chemical reactions between the cementitious materials and mix water. These minerals do not occupy all of the space that was once occupied by water, and thus there is considerable pore space, called capillary pores, that remains in the 0.01 to 5 micron size range (Mindess et al. 2003). Capillary pores have a large influence on concrete strength and permeability. They also have a large influence on the volumetric change that concrete undergoes with changes in moisture. As concrete dries, the pores begin to empty, starting with the largest pores first. In such small pores, as they transition from being full to partially filled with water, a menisci forms due to surface tension. The menisci in millions of pores pull on the pore sidewalls, drawing them in ever so slightly. As drying continues, increasingly smaller pores become partially empty, and the surface tension of the water increases as does the stress pulling the pore walls inward. The net result is concrete shrinks as it dries, thus the need for joints to accommodate the shrinkage, or in the case of CRCP, the relatively high volume of steel to hold the shrinkage cracks that form closely together.

As dry concrete is re-wetted, the opposite occurs. Empty or partially empty capillary pores take up water, the smallest pores first, until full and the menisci disappear. The pore walls rebound but not to the same degree as where they started, as only a portion of the shrinkage is reversible whereas part in irreversible, as shown in Figure 11.4. The net result is that once concrete has dried, it will never return to its original volume as a result of being re-saturated alone.

Figure 11.4 Plot showing shrinkage and partial recovery of concrete due to drying and re-wetting



Recreated from Mindess et al. 2003, ©2003, 1996 Pearson Education, used with permission

Volume change due to temperature, on the other hand, is a reversible phenomenon. The CTE defines how a material changes in length with a change in temperature. It is reported as the change in length of a specimen over the total specimen length per degree change in Fahrenheit (°F) or Celsius (°C). For concrete, CTE typically ranges from 4 to 7.5 x 10-6 inch/inch/°F, with lower values indicating less length change for the concrete for a given change in temperature. As the volume of concrete is predominately aggregate, the aggregate (particularly the coarse aggregate) has a big effect on CTE, with pure limestone aggregates producing concrete with the lowest CTE values whereas concrete produced with siliceous aggregates such as quartzite having the highest CTE values (FHWA 2016).

It has been observed that thermal length change is often complicit in the occurrence of blowups, with increasing frequency as temperatures increases. Notably, blowups are often associated with heat waves, when the existing joints are not able to accommodate the thermal expansion in the concrete. It has been observed in some states, most notably Minnesota, that the risk of blowups is further increased if the heat wave is accompanied by a significant precipitation, which results in saturation and further expansion of the concrete.

The occurrence of blowups from hot temperatures alone is an exception, rather than a rule, because the shrinkage that occurs as concrete dries often is greater than the thermal expansion that occurs during hot temperatures alone. Thus, although thermal expansion is complicit, other factors contribute to the occurrence of blowups.

Prevention

It is important to consider the CTE of the coarse aggregate. A high CTE coarse aggregate will result in greater thermal expansion as the concrete temperature increases. Although this alone will not result in blowups, it can combine with other factors such as high levels of saturation and/or AAR, resulting in a net expansion of the slab and the buildup of compressive stress that may result in blowups. The risk is more acute when high CTE coarse aggregates are used in pavements with long joint spacing.

The time of year during which the pavement was constructed was a factor with long-jointed JRCP but this pavement type is not currently recommended so this factor is less important.

Volume Expansion Due to Aggregate Deterioration

Cause

Volumetric changes due to changes in concrete moisture and temperature can be contributory factors to the occurrence of blowups but are not often the sole cause. In contrast, deleterious physical or chemical reactions that result in expansion of in-service concrete can result in the occurrence of blowups.

For instance, concrete pavements affected by aggregate freeze-thaw deterioration (i.e., D-cracking) or an AAR have been known to undergo sizable expansion that has been linked to blowups, as well as shoving of shoulders and fixed structures such as bridge abutments. ACR is rare but extremely expansive. A pavement affected by ACR will almost certainly experience expansion-related distresses that include blowups. ASR can result in varied amounts of expansion, from benign to excessive. A pavement experiencing ASR should be carefully monitored to determine the magnitude of expansion. In all cases, if AAR is suspected, core samples should be obtained and evaluated to confirm the presence of ACR and/or ASR using petrographic analysis in accordance with ASTM C856, and determine residual expansion to make an assessment whether the AAR has run its course or whether significant expansion has yet to be realized.

Prevention

The primary materials factor that can contribute to blowups is expansion resulting from the use of aggregates that are susceptible to freeze-thaw damage (i.e., D-cracking) or AAR without mitigation. Aggregate sources should be screened for both types of susceptibility. States in which D-cracking is an issue have standard specifications to screen coarse aggregates for this distress. The risk of AAR should be addressed through the application of the AASHTO PP 80 protocols, which provide screening protocols and appropriate mitigation strategies to be applied if ASR susceptible aggregates are to be used. ACR cannot be mitigated and thus ACR susceptible aggregates must not be used.

Design Factors

Cause

Pavement design can play a role in the development of blowups. In general, long JPCP and CRCP will be more susceptible than long jointed reinforced concrete pavement (JRCP). This is related to the movement at individual transverse joints which is greatest for JRCP. During winter, the opening of joints in JRCP will be relatively wide and will allow for the infiltration of considerable amounts of incompressible material if they do not remain sealed. These incompressible materials can prevent the joint from closing adequately during the heat of summer, potentially resulting in sufficient stress to produce blowups.

Prevention

The risk of blowups is significantly reduced if the spacing between and widths of transverse joints are reduced (such as in short JPCPs) or eliminated entirely (such as is the case with CRCP). If the pavement is jointed, the joint reservoir should be carefully designed to be either narrow (single saw cut) and unsealed or designed with a good sealant in mind.

Another design factor that can reduce the risk of blowups is good pavement drainage design. A pavement design that does not allow water to drain from beneath the slab will keep the concrete in or near a saturated state. Although this alone will not result in blowups, it can combine with other factors such as high temperatures and/or AAR, resulting in a net expansion of the slab and the buildup of compressive stress that may result in blowups.

Some states, most notably Minnesota, have observed that good pavement drainage can also reduce the risk of blowups.

Another design factor specific to concrete overlays of concrete pavements is that pressure relief joints that are present in the existing pavement must be matched in the overlay. This will prevent movement in the underlying pavement from generating excessive stress in the overlay that may result in a blowup.

Time of Construction Factors

Cause

In some cases, the time of year in which a pavement was constructed has been linked to the occurrence of blowups. Michigan DOT, for example, used to require that pressure relief joints be an element of new construction of JRCP that was built either early in the spring or late in the year. The rationale for using pressure relief joints for JRCP constructed during the cooler months was that the thermal expansion that would occur when the pavements were subjected to temperature in excess of that experienced during construction would exceed the shrinkage due to drying; the net result being the generation of compressive stress at the joints. Constructing pressure relief joints was thought to alleviate the buildup of this stress and prevent blowups. A negative impact of this practice was the observation that transverse joints on either side of the pressure relief joint opened wider than anticipated, allowing for the infiltration of incompressible materials and resulting in loss of load transfer at those joints and the closure of the pressure relief joint. With the adoption of JPCP, this practice is no longer followed as the smaller joint movement inherent in JPCP is not as susceptible to infiltration of incompressible materials.

Prevention

The time of year during which the pavement was constructed was a factor with long JRCP but this pavement type is not currently recommended, so time of construction is less important.

Incompressible Materials

Cause

The most common cause of blowups is not a design, materials, or construction issue but the filling of transverse joints with incompressible materials once the joint sealant is no longer functional. Incompressible materials will enter the joints during cold winter months when the joints are opened to their widest level due to thermal contraction. Joint width opening increases as joint spacing increases, thus more material can infiltrate the joints of long JRCP than in short JPCP. As the pavement warms in the summer, the incompressible materials restrict joint movement, not allowing them to close. The resulting buildup of compressive stress at individual joints increases the risk of blowups during the hottest and wettest days of the year.

Prevention

The primary maintenance factor that reduces the risk of blowups is to keep pavement joints free of incompressible materials. If the pavement was originally unsealed, the buildup of excessive incompressible materials in the joints may be an indicator that cleaning and sealing/filling the joints may be advisable, particularly if other joint distresses such as spalling are observed. Sealed pavements should be monitored, and if the sealant is in poor condition or if the joints are becoming filled with incompressible materials, the joints should be cleaned and resealed prior to the occurrence of spalling and/or blowups in accordance with procedures identified in (Smith et al. 2014). Table 11.3 provides a summary of the mechanisms leading to curling and warping as well as design, materials, construction, and maintenance factors that can be used to prevent or mitigate it.

Table 11.3 Summary of causes and prevention or mitigation of blowups

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Blowup	Expansion of slab during hot weather is retrained by incompressible materials in joints An expansive reaction (e.g., ASR, ACR) results in slab expansion	Use of short joint spacing typical of JPCP will minimize joint opening Use of CRCP Good drainage can help reduce the risk	Use non-deleterious aggregates to avoid expansion If ASR reactive aggregate must be used, mitigate reactivity using AASHTO PP 80 Low CTE aggregate reduces risk	The time of year in which a pavement is constructed was a risk factor for blowups for long JRCP constructed in the spring or fall but with the use of short JPCP, this risk has been addressed	Keep transverse joints cleaned and sealed, keeping incompressible materials out Inspect and maintain drainage system if one exists If expansion of the concrete is the issue, pressure relief joints can be cut as a stopgap measure to maintain short-term serviceability

6. Treatment and Repairs

Blowups are caused by multiple factors that often combine to create excessive compressive stress in the slab that is relieved when the concrete crushes and/or thrusts upward, requiring immediate repair to restore serviceability. The following summarizes strategies to prevent blowups from occurring, treatments to be applied to pavement that's prone to blowups, and techniques used to repair a blowup once it has occurred.

Treatment

If a pavement has suffered blowups, the cause must be definitively identified. The two primary causes are the presence of AAR or infiltration of incompressible materials into the joints.

Treatment of AAR Affected Pavement

There are no effective treatments to stop D-cracking or AAR once it is occurring in a pavement. If either is suspected, a petrographic analysis conducted in accordance with ASTM C856 should be used to determine the nature of the deterioration mechanism at work.

If ACR is identified, it is very likely that significant expansion will continue. This pavement will likely require reconstruction. There are no strategies to mitigate ACR, and thus, the ACR susceptible aggregate should not be recycled back into a cementitious material. As a stopgap measure, pressure relief joints should be installed to specifically relieve the buildup of compressive stress (FHWA 1990). Typical pressure relief joints are cut 4 inches wide every quarter of a mile. To prevent blowups in adjoining lanes, pressure relief joints should be the full width of the entire roadway. If done as reactive maintenance to address a rapidly developing blowup problem, it is best to make the relief cut early in the morning during the coolest time of day as the pressure will be the least. Pressure relief joints should be closely monitored as they will likely compress over time, requiring maintenance to restore ride quality. Additional pressure relief joints will need to be installed as required.

If D-cracking or ASR is identified, it is possible that the expansion will slow or even cease over time. In the case of ASR, residual expansion testing of extracted cores can be done to make this assessment. Regardless, pressure relief joints can be installed into the affected pavement to relieve the buildup of compressive stress. All pressure relief joints will need to be monitored and maintained, and additional joints may need to be installed.

It is stressed that the installation of pressure relief joints is a last resort, stopgap measure to maintain serviceability in the interim before a more permanent solution can be implemented.

Treatment of Incompressible Materials

Incompressible material in joints can only be treated by cleaning the joints and resealing them. Procedures for joint resealing can be found in Smith et al. (2014). Joint cleaning and resealing should be done as soon as incompressible material is observed in joints, and ideally prior to the development of blowups. If blowups are occurring, joint cleaning and resealing alone might not prevent the occurrence of future blowups and pressure relief joints may need to be installed.

Repair

Blowups must be repaired to restore pavement functionality. The most common repair strategy is to construct a full-depth repair in accordance with guidance provided in the *Concrete Pavement Preservation Guide* (Smith et al. 2014). Repair boundaries must be selected to encompass the entire failed area, which may be of greater extent below the slab. For JPCPs, the transverse joints of the repair should be dowelled to the existing pavement, whereas for CRCP, the steel must be restored across the repair prior to placement of the concrete.

Depending on slab size and extent of damage, full panel replacements may be needed to repair a blowup. These are constructed similarly to a full-depth repair but the full panels before and after the blowup are removed in their entirety and new concrete panels match existing joints. The use of precast panels or accelerated paving mixtures can be employed to expedite time to opening if needed.

Overlay

Prior to overlaying a pavement prone to blowups, the cause of the blowups should be definitively determined. If the expansion is resulting from ACR, overlaying is not recommended as it will not prevent continued expansion. Overlaying may even accelerate it by increasing the moisture content in the underlying concrete. If ASR is identified as the cause of expansion, great care should be exercised in using an overlay as a rehabilitation strategy. Residual expansion testing should be conducted to assess the potential for future expansion, recognizing that an overlay will likely increase the moisture content in the underlying concrete, and thus may accelerate the reaction and increase the risk of future blowups.

If blowups are a result of incompressible materials in the joints, the joints should be cleaned prior to overlaying. Thicker overlays reduce the temperature that the underlying slab will experience on hot days, thus reducing thermal expansion.

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CHAPTER 12. SUBGRADES AND BASE SUPPORT CONDITIONS—CATEGORIZED AS SETTLEMENT AND HEAVES DISTRESSES

1. Description

Other than vehicle loading, the principal cause of distress in concrete pavements is volume change, or put another way, movement. This can occur in either the concrete itself or in the underlying support system. The volume changes result in movements in the concrete, which either exceeds the design parameters of the pavement when distresses occur or when not anticipated in the design. The volume change in concrete itself has been covered in other chapters of this manual. This chapter is devoted to the distresses in the concrete pavement due to volume changes in subgrades and bases. (See Figure 12.1)

The distresses from volume change (or movement) in concrete pavement from the subgrade and base typically show up as cracking as the result of settlements or heaves of the subgrade/base. Therefore, this chapter has been developed and formulated around settlement and heave categories since they do represent the majority of subgrade and base distresses.

The mechanisms for settlements and heaves are very different, even though they are both dependent on movement within the subgrade or base course. As such, each of these distress types will be addressed separately, where appropriate. Settlements in concrete pavements can be described as localized downward slab displacements, while heaves are defined as upward localized slab displacements. In both cases, this distress type can be characterized as differential vertical displacement between adjacent slabs or within cracked slabs. This does not mean a horizontal movement of the cracked pavement is caused by some other phenomena. It is still caused by volume change of the underlying support system, which happens to be greater in the horizontal direction than in the vertical direction; however, there is still vertical movement taking place.

Settlements and heaves affect ride quality and long-term pavement performance, and they are considered to be functional distresses in all cases. However, they are frequently accompanied by random cracking, which is indicative of a more critical structural distress (ACPA 2007). Displacements may range from almost imperceptible elevation differences to several inches or more in rare cases. The accompanying distresses (primarily cracking and loss of ride quality) are correlated to this degree of movement.

Faulting at joints or cracks is generally considered as a separate distress category although differential vertical movement is present. Note that traffic loading is not necessary for settlements to occur, as in faulting but may be a contributing factor in some cases. Heaving is not influenced by traffic loading although the corresponding distresses, such as random cracking typically are.

Figure 12.1 Various settlements and heaves

a. Settlement due to moisture-related subgrade soil movement
Michael Ayers, Global Pavement Consultants, Inc.

b. Cracking due to trench backfill settlement
Michael Ayers, Global Pavement Consultants, Inc.

c. Pavement settlement adjacent to a rigid structure that has not settled

Dale Harrington, HCE Services

d. Corner break and settlement due to subgrade soil movement under traffic

Michael Ayers, Global Pavement Consultants, Inc.

e. High severity cracking due to frost heave of the subgrade soil
Michael Ayers, Global Pavement Consultants, Inc.

f. Settlement due to subgrade soil movement under traffic
Dale Harrington, HCE Services

g. High severity cracking due to settlement and loss of support
Tom Burnham, Minnesota DOT

h. Settlement due to consolidation of the subgrade soil without significant traffic loading
Dale Harrington, HCE Services

2. Severity

Pavement distresses associated with settlement and heaves typically include random cracking (longitudinal, transverse, diagonal, and shattered slabs) and localized or overall pavement roughness. These distresses, with corresponding severity assessments, are included in the Identification of Causes Section of this chapter.

Settlements and heaves are not characterized according to the *Distress Identification Manual for the Long Term Pavement Performance Program* (Miller and Bellinger 2014). In general, the severity of settlements and heaves are not reported, as they commonly occur with other types of pavement distresses due to the fact that any appreciable slab movement results in nonuniform pavement support. However, the likely cause of the distress, as well as the differential movement along with the extent of the distress as shown in Table 12.1, may be reported in order to help to determine repair quantities and appropriate remediation.

Table 12.1 Summary of severity levels and measurement

Distress	Description	Severity Levels	Measurement
Settlements	Settlements are localized downward differential displacements that occur between slabs or within a cracked slab. Settlements are generally accompanied by increased pavement roughness and slab cracking.	The following severity levels pertain to the measurement of individual settlements. **Low**: settlement of less than or equal to 1/2 in. (13 mm) **Medium**: settlement of 1/2 in. (13 mm) to 1 in. (25 mm) **High**: settlement of greater than 1 in. (25 mm)	Surface area of settlement Average vertical displacement within the boundaries of settlement Other distresses should be measured based on guidance provided elsewhere in this document Ride quality measurements may be required in areas exhibiting widespread settlements
Heaves	Heaves are localized upward differential displacements that occur between slabs or within a cracked slab. Heaves are generally accompanied by increased pavement roughness and slab cracking.	The following severity levels pertain to the measurement of individual heaves. **Low**: heave of less than or equal to 1/2 in. (13 mm) **Medium**: heave of 1/2 in. (13 mm) to 1 in. (25 mm) **High**: heave of greater than 1 in. (25 mm)	Surface area of heave Average vertical displacement within the boundaries of the heave Other distresses should be measured based on guidance provided elsewhere in this document Ride quality measurements may be required in areas exhibiting widespread heaves

3. Testing

Determining the cause and most feasible rehabilitation option for both settlement and heaves requires evaluation of the existing pavement support conditions. Testing may include both destructive and/or nondestructive methods depending on the size, relative importance, and location of the project. The overall goals of the testing program are to assess the current support values provided by the foundation layers, determine the reason for the movement causing the distresses, discover the extent of the problem, determine if the pavement has voids under it or nonuniform support to the extent that continued movement is likely, and finally, to develop effective repair strategies. The following are the tests that should be considered depending on the pavement condition. See Chapter 19 for more details.

Field Tests

Falling Weight Deflectometer (FWD)

The FWD can be used to estimate the level of support under the slabs and differences in support as is typical for both settlements and heaves.

Coring and Material Sampling

The most appropriate method to determine the cause and extent of settlement and heaves is to core the pavement and extract material samples of both the base (if present) and subgrade soil. These samples will then be tested in the laboratory to determine relevant material properties including particle size analysis, Atterberg limits (plasticity index, shrinkage limit, plastic limit, and liquid limit), in situ moisture content, and other applicable tests.

Dynamic Cone Penetrometer (DCP)

The DCP can be used to rapidly estimate the California Bearing Ratio (CBR) or other soil strength/deformation parameters of the base and subgrade layers. Prior to sampling the base/subgrade, the DCP tests can be conducted through the core holes and to a typical depth of 3 to 4 feet.

Plate Load Testing (PLT)

Static plate load testing can be used to directly measure modulus of subgrade support, or "k" (AASHTO T 222), or resilient modulus (M_r) can be directly measured using repetitive/cyclic testing (AASHTO T 221).

Ride Quality Measurement

Ride quality measurements can provide useful information on the extent of the settlement or heaved area. It is important to determine if the displacements are localized or a more global type of distress when selecting the most viable prevention or remediation technique. If only localized heaves or settlements are present, a straightedge may provide sufficient information. If a more global distress is present, a profiler is a more appropriate tool for data collection.

Laboratory Tests

Laboratory testing provides useful information relating to the cause and potential remediation measures of both settlements and heaves. Appropriate tests vary somewhat based on the assumed cause. For instance, was the heave likely due to the formation of ice lenses (frost heave) or a high sulfate content in the soil if a cement stabilized base was used?

The tests that should be routinely performed include gradation or particle size analysis, moisture content, and Atterberg limits (AASHTO soil classifications T 89 and T 90). These values should be determined and compared between areas experiencing movement and those with no movement. Additional tests to be performed depend on the site conditions and potential causes of the settlements or heaves. For instance, if evidence exists of expansion within a stabilized base layer, it is necessary to determine the amount of sulfates present in the underlying soil as well as the groundwater.

The AASHTO soil classification system is one of the most widely used in estimating materials properties of naturally occurring granular materials to fine-grained soils. Many references regarding settlements and heaves reference the AASHTO classification, as shown in Figure 12.2.

Figure 12.2 AASHTO soil classifications

GENERAL CLASSIFICATION	**GRANULAR MATERIALS (35 percent or less of total sample passing No. 200 sieve (0.075 mm)**							**SILT-CLAY MATERIALS (More than 35 percent of total sample passing No. 200 sieve (0.075 mm)**			
GROUP CLASSIFICATION	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, percent passing:											
No. 10 (2 mm)	50 max.										
No. 40 (0.425 mm)	30 max.	50 max.	51 min.								
No. 200 (0.075 mm)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No 40 (0.425 mm)											
Liquid limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
Group Index**	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.

Classification Procedure: With required test data available, proceed from left to right on chart; the correct group will be found by the process of elimination. The first group from the left into which the test data will fit is the correct classification.

*Plasticity Index of A-7-5 subgroup is ≤ LL minus 30. Plasticity Index of A-7-6 subgroup is > LL minus 30.

**Based on group index formula (equation 4-3) in AASHTO M 145.

Group index should be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-7-5(17), etc.

Based on AASHTO M 145-91 (2004) (or ASTM D382), *Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes*, Table 1 and Table 2, ©2004 American Association of State Highway and Transportation Officials, Washington, DC. Used with permission.

The AASHTO Soil Classification System was developed by the American Association of State Highway and Transportation Officials, and is used as a guide for the classification of soils and soil-aggregate mixtures for highway construction purposes. The classification system was first developed by Hogentogler and Terzaghi in 1929 but has since been revised several times.

The characteristics of soil can be discovered with the soil classification system. The typical problem of soil settlement and heaves begins with silts, plastic, and organic clays, along with poorly graded soils with poor stability. Clays with low shrinkage limits continue to dry, resulting in continuing shrinkage and cracking of the subgrade. Clays with low plastic index take only a small amount of moisture to cause them to change from a plastic condition to a liquid condition. Clays with a high plastic limit and index have a higher swelling potential. Finally, soils with a higher liquid limit have a greater potential for compressibility. Therefore, it is important to have an understanding of how soil properties can affect the performance of the subgrade.

4. Identification of Causes

The causes of settlements are generally due to nonuniformity and inadequate compaction of the support layers during construction, or consolidation of these layers under traffic or erosion of the base/subgrade. Heaves are typically due to the formation of ice lenses (frost heave), swelling soils, and in some cases, expansion of a stabilized base material due to adverse chemical reactions. A summary of causes is presented in Table 12.2.

Table 12.2 Summary of causes of pavement settlements and heaves

Distress	Category	Description
Settlement (Physical)	Inadequate drainage system	Poor drainage of the subgrade causes erosion and loss of uniform support, pumping, and isolated voids Where soils dictate, such as silts and clays, place proper drainage base to provide a minimum uniform support system Over time, certain silt and clay subgrade soils can intrude into and contaminate the base layer
Settlement (Physical)	Inadequate subbase and base compaction during construction	Inadequate compaction can lead to consolidation under traffic, or natural consolidation Isolated spots of hard structures such as manholes, intakes, pipes, etc., which do not settle equivalent to the adjacent slab and gives the appearance of heave of the structure
Settlement (Physical)	Inadequate trench compaction	Adequate compaction of trench backfill material is difficult to achieve and typically leads to differential compaction relative to the adjacent areas
Settlement (Material)	Variable subgrade soil types	Nonuniform subgrade soils types have different absorption, exposure, shrinkage characteristics; i.e., they move differently, causing differential movement in the pavement and cracking
Settlement (Material)	Local consolidation of base and/or subgrade under traffic	Soils prone to volume changes as a function of moisture and density variations can undergo continued consolidation under repeated load applications Granular base materials with low abrasion resistance can breakdown under repeated loading Differential or inadequate compaction, poor gradation control, high fines content (particularly materials with high plasticity index); can result in areas of variable consolidation and settlement

Distress	Category	Description
Settlement (Material)	Trench backfill materials	Improper gradation control, particularly high fines content, can lead to moisture sensitivity and differential consolidation and settlement
Settlement (Material)	Subgrade soil movement	Collapsing soils are those that experience large volume changes as the moisture content of the soil increases significantly
Heaves (Physical)	Formation of ice lenses in the subgrade (frost heave)	Ice lenses form in certain types of soils when sustained freezing temperatures are present and accompanied by high soil moisture content Frost susceptible soils, through their grain size distribution, are capable of wicking up the moisture necessary for the formation of ice lenses
Heaves (Physical)	Expansive (swelling) subgrade soils	Expansive soils are those that expand or swell in response to increasing moisture content in the soil Expansive soils undergo a change in basic structure as a function of increased moisture content

5. Evaluation

Settlements and heaves in concrete pavements are due primarily to volume changes within the subgrade soils and unbound base materials. Settlements are generally driven by traffic loading, although, on occasion, they can occur without load applications. Heaves are independent of traffic loading and are strictly an expansion of the subgrade soil or in the case of blowups, expansion of concrete. It should be noted that bases/subbases can also be subject to settlements and heaves if there are sufficient fines (minus No. 200) present.

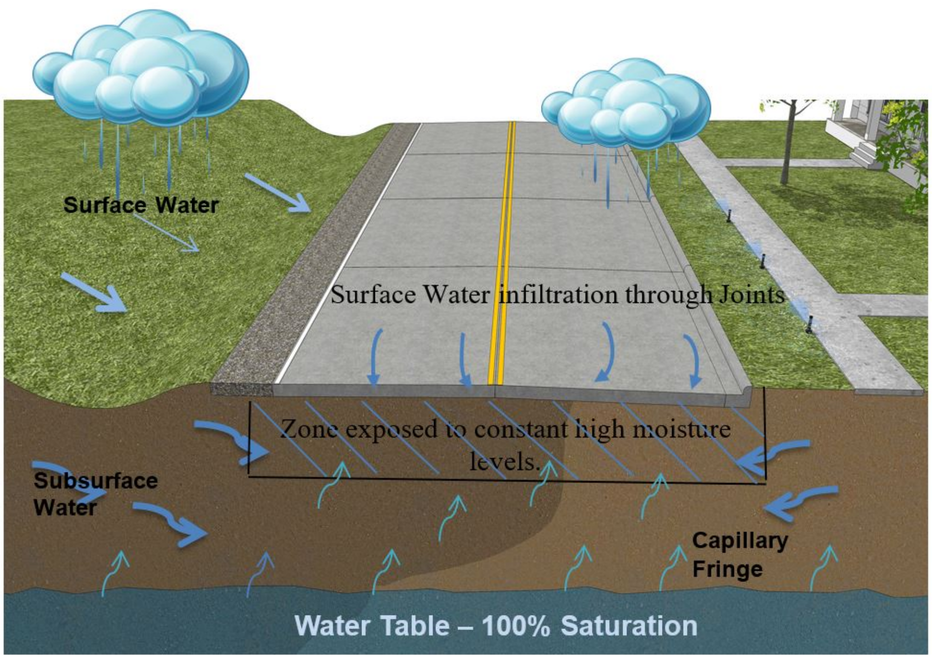
Settlements and heaves happen for a variety of reasons, as shown in Tables 12.2 and 12.3. The information provided in these tables summarizes the most common causes; however, it should be noted that a combination of these factors may be responsible for the observed pavement distress. Rehabilitation options generally entail removal and replacement of the affected pavement slab(s), as well as remediation of the pavement support layers. In the case of settlements, slab jacking may be a viable remediation option. Selection of the most cost-effective rehabilitation option requires a detailed evaluation of the causes of the settlements and heaves.

Bases or subbases are sometimes omitted in the design of low and moderately trafficked pavements. This decision is generally based on economic considerations and is justified on the basis of a perceived minimal contribution of these layers based on current design methods. However, historical performance data indicate that the inclusion of a base can greatly enhance long-term pavement performance. The function of the base/subbase layers is to provide a construction platform, increase support, facilitate drainage (if designed as a drainable layer), and to reduce subgrade stresses. Experience has shown that the majority of settlements are due to nonuniform subgrade soils, lack of compaction during construction, and inadequate drainage of the foundation layers. In addition, settlements are oftentimes driven by an increase in the moisture content of the subgrade soil. Providing adequate drainage should be considered during design if the soil type has been shown to be moisture sensitive. Heaves are due to a combination of moisture and temperature effects. A frost susceptible soil, lack of adequate drainage, and/or nonuniformity of the subgrade lead to differential frost heave.

Inadequate Drainage System

A major cause of settlement and heave is the result of an inadequate drainage system for the concrete pavement system. Water migrates into the pavement structure through a combination of surface and subsurface infiltration, edge inflows, and from the underground water table (e.g., via capillary potential in fine-grained foundations such as clays and silts). In cold environments, the moisture may undergo seasonal freeze/thaw cycles. As shown in Figure 12.3, the collection of this high moisture level can concentrate in a zone below the pavement. It is for this reason that a proper subsurface drainage system, as shown in Figure 12.5, needs to be considered to help intercept and outlet the subsurface water before it concentrates under the pavement. This prevents the underlying support system from collecting water which results in saturation, loss of strength, erosion of base material, and thus, pavement distresses such as settlement and heaves, and subsequently, cracking.

Figure 12.3 Concrete pavement exposed to surface and subsurface moisture conditions



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Cause

Moisture within the pavement system, particularly in wet and cold weather states, nearly always causes detrimental effects on pavement performance via settlement and heaves. It reduces the strength and stiffness of the pavement foundation material, results in differential volume change of soil, promotes contamination of granular subbase material due to fines migrations, and can cause swelling of plastic soils (e.g., frost heave and or soil expansion) and subsequent consolidation. Contamination may also be due to improper materials, fines generated during placement and compaction, abrasion under traffic, intrusion of subgrade soils into more open graded materials, and breakdown of the aggregates due to wetting/drying or freeze/thaw cycles.

With an inadequate drainage system, contamination of the base over time is highly likely and results in pumping of subsurface water, and subgrade/subbase fines (See Figure 12.4). This loss of fines results in voids and causes cracking and settlement.

Figure 12.4 Pumping evidence of saturated subgrade or base causing pumping through the transverse joint

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Prevention

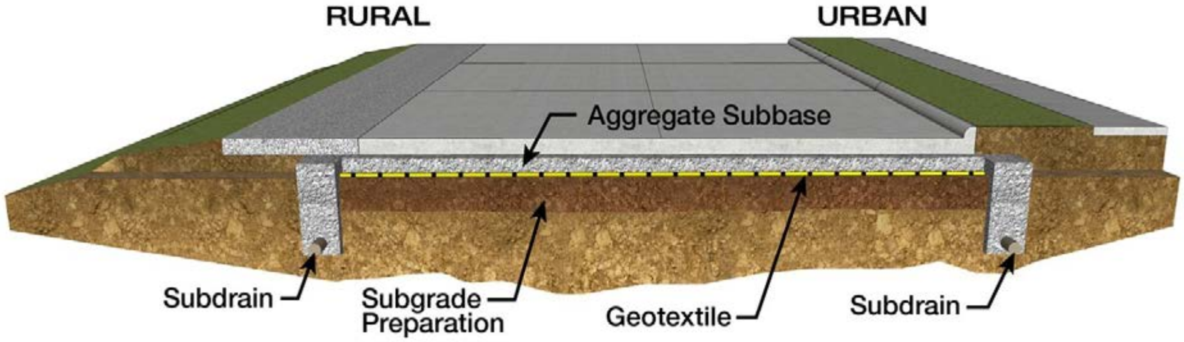
All concrete pavements should be constructed on a uniform, drainable, and stable support system.

To help minimize settlement and heaves associated with moisture-related conditions, a major objective of pavement design should seek to prevent saturation or exposure to constant high moisture levels of the subgrade 2 to 3 feet underneath the pavement, including the base. This can be accomplished by considering the following approaches.

Natural Subgrade, No Base: Certain natural soils can provide the uniform, drainable, and stable support system for concrete pavements, particularly for low-volume roads. These soils consist of a well-graded course to fine granular material (A-1), or granular material with some binding characteristics (A-2). Even silts (A-4) or clays (A-6) can perform well enough for local roads without a base when they consist of some granular or sandy material, have low plastic characteristics, are well graded, have adequate density, and have a low affinity for water. However, these characteristics are rare, and in a number of cases, the soil type is constantly changing, which results in differential movement and nonuniformity.

Unstabilized Aggregate Base: As shown in Figure 12.5, a common and cost-effective approach is to utilize an unstabilized aggregate base for these nonuniform but suitable soils. An aggregate base provides an outlet of surface water draining through the pavement joints and helps reduce shrink and swell of high volume-change soils. It minimizes mud pumping of fine grain soils and prevents consolidation of the subgrade. It also provides a capillary cutoff for higher water table conditions when silts and clays are part of the subgrade. Finally, the aggregate base provides a working platform during construction as long as there is a proper distribution of aggregate sizes to allow for stability and drainage at the same time. The maximum particle size should be no more than one-third of the base thickness, less than 10 percent passing the No. 200 sieve, maximum relative density of 70 percent, plastic index of 6 or less, and liquid limit of 25 or less. A target permeability should be about 150 feet per day.

Figure 12.5 Typical subsurface drainage system



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Stabilized Soil or Stabilized Base: Those soils that have high moisture content or high expansive/shrinkage characteristics with low density (less than 95 pounds per cubic feet) will typically require either additional soil modification and/or a stabilized base. See "Formation of Ice Lenses in the Subgrade (Frost Heave)" and "Expansive Subgrade Soils" of this chapter for additional details.

Subdrain: To provide an outlet for an aggregate base, a subdrain system should be utilized, preferably on both sides of the roadway. If it is desired to place a subdrain on only one side of the roadway, a number of factors need to be considered before making such a decision. These include redundancy precautions, whether or not the roadway in a full cut section, the groundwater conditions, whether or not there will be a watering system on both sides of the roadway in urban areas, etc. The location of the outlet is extremely important in order to provide proper maintenance on a routine bases. Where possible, it should be outleted to a structure such as a manhole or intake.

Capillary Break or Cutoff: When subgrades consist of silts and clays (fine soils with high capillary potential), a capillary break or cutoff should be placed between the aggregate base and subgrade. The purpose is to prevent the migration of fines into the aggregate subbase, resulting in the loss of uniform support to the pavement and the consolidation of the subgrade. The capillary cutoff can consist of a filter layer such as a geotextile woven fabric or a 4-inch plus or minus dense-graded aggregate layer.

Table 12.3 Subgrade treatment based on condition of the subgrade

No.	Subgrade Conditions	Treatment
1	Varying types of soil in subgrade but meets moisture and density control conditions and passes proof rolling test	Disk and mechanically blend the soils at 8 in. lifts to 2 ft deep, particularly when pavement is placed on natural subgrade without a base. Compaction should be at 95% standard Proctor density.
2	Uniformly wet soils and will not pass proof rolling or density test	Dry the subgrade by disking during drying weather. If the drying weather window is not available or the soils are too wet to dry, utilize quick lime, cement, or fly ash to dry the subgrade.
3	Expansive or unsuitable soils	Chemically stabilize the soil with cement, changing the shrinkage, plastic, and liquid limits to acceptable levels. A more extensive approach is to remove the unsuitable soils and replace with select material.

Inadequate Subgrade and Base Compaction During Construction

Lack of uniform and adequate compaction during initial construction is one of the most frequent causes of settlement in concrete pavements. The soil classification and engineering properties of the in situ subgrade soils are generally determined during the project feasibility stage and used later in pavement design. (See Table 12.3.) Traditionally, field density testing (at optimum moisture) has been used to evaluate whether adequate support has been achieved. Field moisture and density (referred to as M&D) is based on the standard or modified Proctor density test, which relates compacted density to the moisture state of the soil and compactive effort. In practical terms, this equates to compacting the subgrade soil at the optimum moisture content using the proper rolling equipment and number of passes determined by test strips. With suitable and uniform soils, this is an important test to achieve the highest strength and stiffness. With poorer soils that have low density (below 100) and a high plastic index, achieving the best compaction possible through proper M&D still results in poor strength. To be able to use these poorer soils, it will take soil modification with cement, fly ash, etc. The important strength and deformation characteristics of the subgrade should relate back to the support values assumed during design. It is critical for long-term pavement performance to achieve during construction the support conditions upon which the design is predicated. These are typically a given stiffness, uniformity of support, and adequate drainage.

Therefore, in design, it is not density that is important but rather the stiffness strength and uniformity of the support layers. Density has historically been used as an acceptance criteria as it has been a practical and relatively easy test to run.

Modern pavement design methodologies are based upon achieving modulus of subgrade reaction (a “K” value) or resilient modulus (M_r) value. Both of these values are derived from plate load testing, where in simple terms, a load is applied to a plate and the response of the foundation layer to that given load is measured. The modulus of subgrade reaction is derived from static testing and resilient modulus is derived from repetitive cyclic testing. Because of the difficulty in running these tests in the field, laboratory derived values are normally used. Advancements in testing technologies, however, now permit direct in situ testing in the field using plate load testing. See Figures 12.6 and 12.7. State highway agencies are beginning to transition to construction requirements based upon stiffness measures.

Figure 12.6 Automated plate load testing equipment

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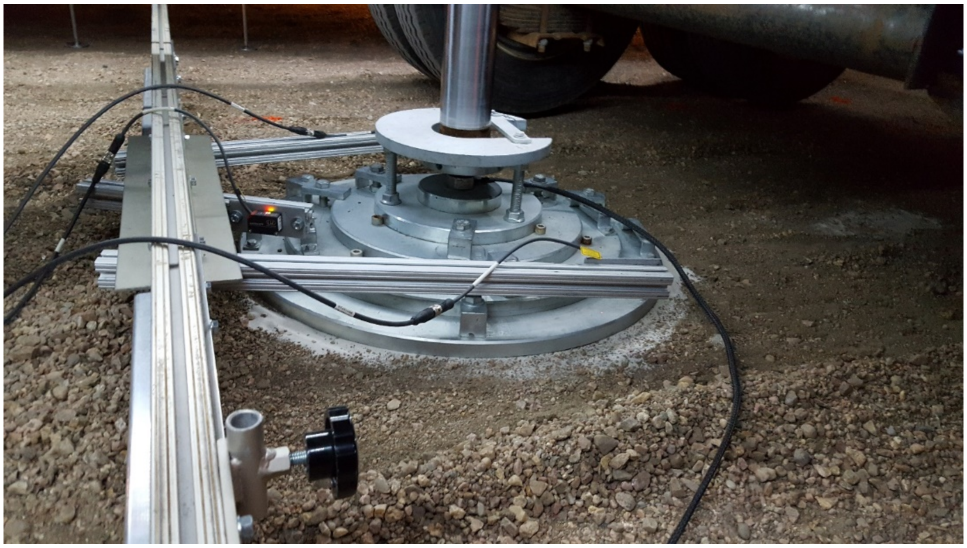
Figure 12.7 Automated plate load testing to directly measure modulus of subgrade reaction and resilient modulus



Ingios Geotechnics, Inc.

Failure to meet the compaction criteria results in a soil that is more deformation prone, leading to long-term settlement. It should be noted that inadequate compaction and segregation of the granular base/subbase layers can also result in settlement.

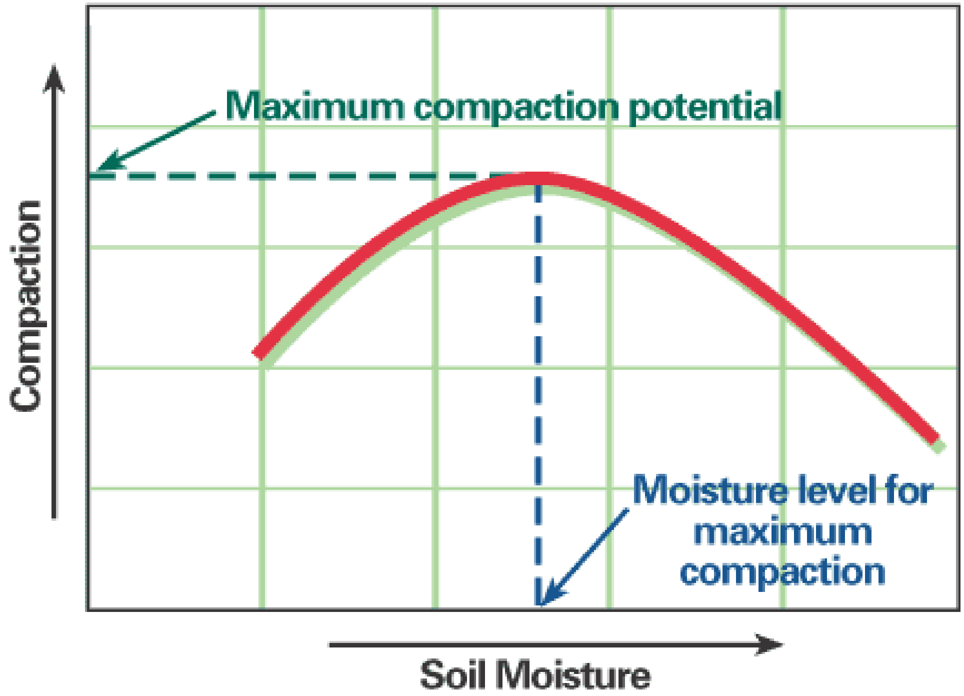
Cause

Inadequate compaction of the subgrade during construction (failure to provide adequate stiffness) may be due to a number of factors including compacting at an inconsistent or incorrect moisture content, use of the wrong type of compaction equipment, or inadequate number of roller passes.

Inadequate base course compaction is generally attributed to a soft or yielding subgrade, incorrect equipment selection, too great of a lift thickness, aggregate gradation, moisture content, segregation of the base material, and an inadequate number of roller passes. See Figure 12.8 for various compaction verification methods.

Figure 12.8 Compaction verification

a. Nuclear density gauge to determine in-place density
Michael Ayers, Global Pavement Consultants, Inc.



b. Moisture-density curve for subgrade soil
Michael Ayers, Global Pavement Consultants, Inc.

c. Proof rolling used to determine weak areas in the subgrade and/or base/subbase
Michael Ayers, Global Pavement Consultants, Inc.

d. Sheepsfoot roller compacting subgrade
Michael Ayers, Global Pavement Consultants, Inc.

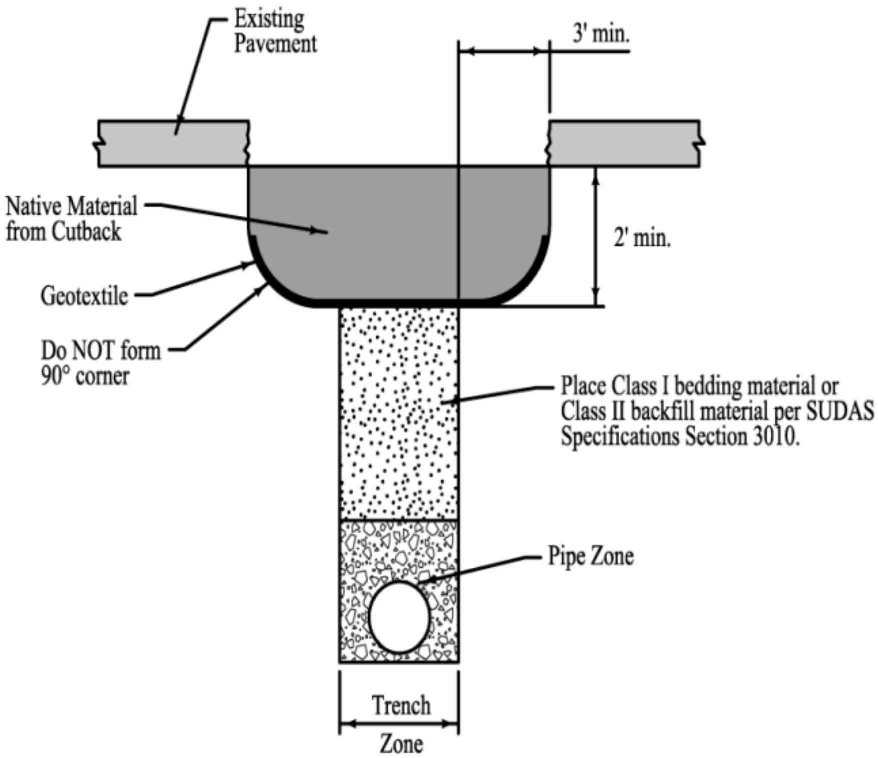
Prevention

Prevention of inadequate compaction requires addressing the stated issues at the time of construction. Close monitoring of the moisture and in-place stiffness is required in all cases. As an added measure, proof rolling of the subgrade and base course will identify areas of inadequate compaction. These "soft" areas should be repaired by additional compaction or, if necessary, removed and replaced with improved materials.

Inadequate Trench Compaction

Trench backfill materials, typically used in urban areas for buried utility placement, are difficult to compact to the same level of density as the surrounding soil and base/subbase. The resulting differential support is problematic for a number of reasons, including a likelihood of longitudinal cracking and settlement. One approach to standardization of trench backfill methodology is the Iowa Statewide Urban Design and Specifications (SUDAS). See Figure 12.9. SUDAS provides details on proper construction techniques to minimize the likelihood of differential compaction. The detailed specifications presented in the SUDAS document include both materials selection and placement techniques to ensure adequate compaction.

Figure 12.9 Flowable fill-in trench (controlled low strength material) backfill being placed for utility cut in pavement



Iowa SUDAS Program

Cause

Trench backfill materials are rarely compacted to the same level of density as the in situ base course materials. The primary reason for this variation is the difficulty in controlling lift thickness and compacting backfill materials in a confined space where conventional rollers cannot be used. Vibratory plate compactors may do an adequate job of compacting these materials but generally do not achieve the same density as the larger equipment. An additional limiting factor is that the compaction energy must be controlled carefully in order not to crush the pipe or damage the conduit.

Prevention

A number of methods have been evaluated in order to provide uniform pavement support at trench locations. In-place trenches during initial construction are generally compacted at a higher level than those placed after the pavement has been constructed (utility repairs for instance). A potential solution for trench backfill is the use of controlled low-strength material (CLSM), a cement-based low-strength fill as shown in Figure 12.10.

Figure 12.10 Backfill option to prevent a stiffer utility trench than the adjacent soil

Michael Ayers, Global Pavement Consultants, Inc.

This self-leveling “flowable” fill can be tailored to the specific application and strength and provides a stable platform for the pavement. Flowable fill compressive strengths should be limited to approximately 100 psi to 200 psi in order to facilitate future excavation. Care must be exercised to not create a hard vertical column that is far stiffer than the adjacent soil. This results in the appearance of a heaved patch, when in reality the utility trench is stiffer as compared to the natural soils changes (See Figure 12.1c as example). Note that in some cases, a reinforcing mat or reinforcing bars are used in the concrete repair to minimize patch deterioration should cracks develop.

Variable Subgrade Soil Types (Pavement Placed on Natural Subgrade)

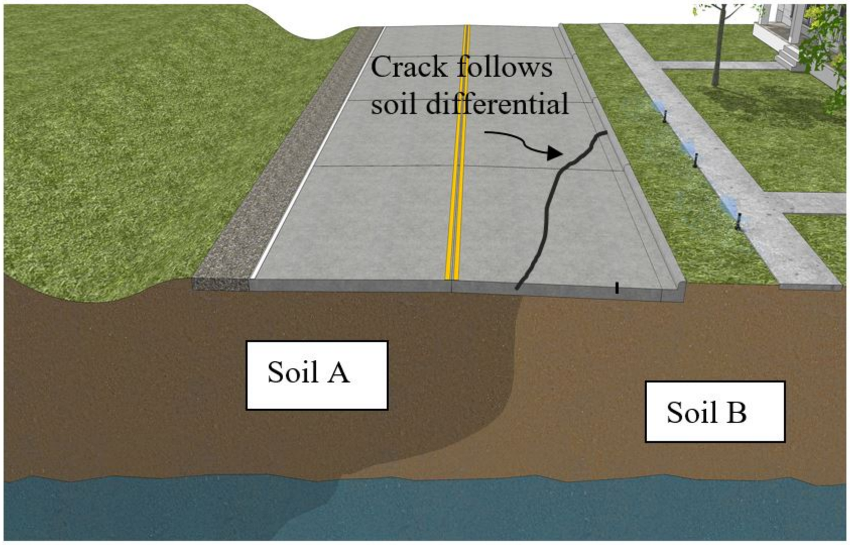
Typical roadway projects are constructed over a variety of soil types, usually with highly variable engineering properties. Concrete pavements, including continually reinforced concrete pavement (CRCP) are designed for a specific strength/deformation (k value) and perform best when constructed on uniform support.

Nonuniform soils provide nonuniform support and may have a tendency to differentially consolidate under traffic as well as undergo volume changes as a function of moisture content. Silty soils may also undergo substantial expansion (frost heave) under the right conditions (adequate moisture and sustained freezing temperatures).

Cause

The simplest form of settlement and heaves, and thus cracking, is when the concrete pavement is placed on natural subgrade (no base with different soil types). The reason for this is that the characteristics of each soil type are different and therefore they absorb moisture differently; i.e., soils shrink differently and have differential volume change. This causes nonuniform support, resulting in cracking along the path of the different soils (see Figures 12.11a and 12.11b).

Figure 12.11 Concrete placed on natural subgrade



a. Different soil types results in differential movement (volume change) of soil due to different soil characteristics of each

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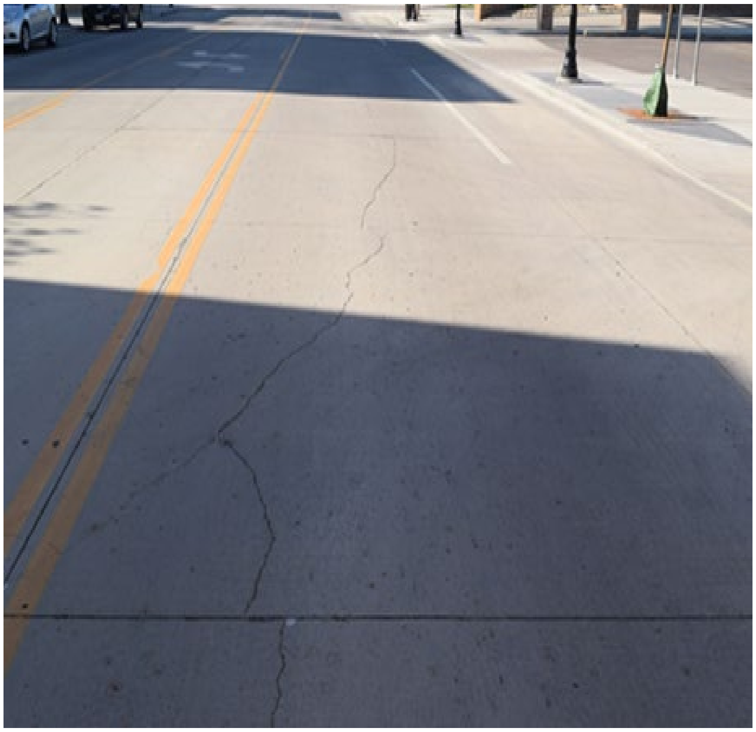

b. Eventual cracking of the concrete pavement

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Prevention

A large number of local concrete roads in the US are constructed on natural soils without a base. To prevent settlement and heave cracks, the subgrade should either be scarified and blended to try to minimize differential volume changes or unbound aggregate base constructed to ensure the uniform platform. See inadequate drainage systems for more details on the benefits of aggregate bases.

Localized Consolidation of Support Layers

Localized and uneven consolidation of the support layers, resulting in localized settlement, can occur for a number of reasons. This type of consolidation is typically associated with either long-term loading (older pavements) without a base or high repetitions of dynamic and heavy loads resulting in high deflections in the concrete slabs.

Cause

Most localized consolidation occurs from nonuniform subgrade soil without a base, which is related to inadequate compaction or inherently weak subgrade. An example is shown in Figure 12.12. This type of consolidation can also occur from loss of unbound aggregate base layers through soil migration. Consolidation of the support layers under traffic can occur in thinner pavements constructed over fill sections, non-optimized base course gradation, and/or poor gradation control. In all cases, the fundamental reason for the settlement is lack of consolidation and nonuniformity of any of the support layers during initial construction.

Figure 12.12 Localized settlement due to subgrade movement (consolidation)

Dale Harrington, HCE Services

Prevention

Proper and uniform placement and consolidation of each of the pavement support layers, as previously outlined, is one of the keys in minimizing the likelihood of continued local consolidation. The other key is to place a uniform base material, such as a drainable aggregate base, for a long-term, uniform, and stable system. To ensure a base will last the design life of the pavement, consideration should be given to placement of a capillary break or cutoff between the aggregate base and subgrade. The purpose is to prevent the migration of fines into the aggregate subbase, resulting in the loss of uniform support to the pavement and the consolidation of the subgrade. The capillary cutoff can consist of a filter layer such as a geotextile woven fabric or a 4-inch plus or minus dense-graded aggregate layer.

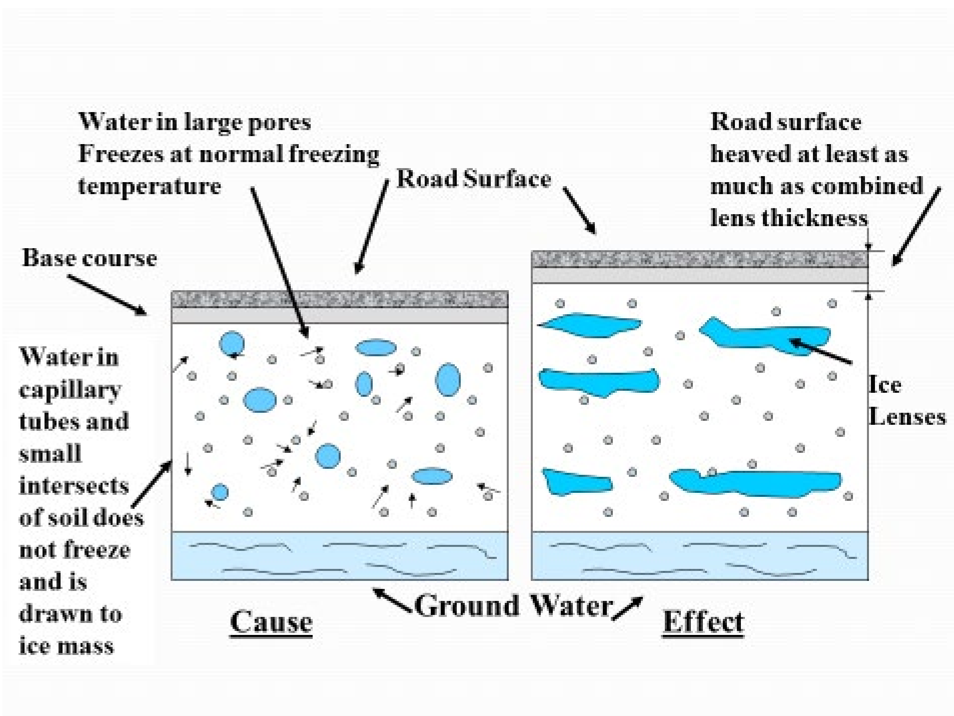
Formation of Ice Lenses in the Subgrade (Frost Heave)

Frost heave is a common type of subgrade soil expansion but it only occurs in areas with sustained freezing temperatures. The amount of heave is dependent on the soil type, its engineering properties, and the amount of free moisture present, typically from groundwater. Frost action includes effects of both the frost heave and the subsequent subgrade softening. Soil types most susceptible to frost heave are fine-grained soils with a high percentage of silt particles or typically classified in the AASHTO A-4 groups. These soils have poor sizes small enough to develop capillary potential large enough to prevent water to travel to the frozen zone (Bly et al. 2008). Coarser soils accommodate higher rates of flow but lack the capillary potential to lift moisture from the water table. More cohesive soils, although they have high capillary potential, have low permeability, which prevents water from moving quickly enough to format ice lenses in the soil.

Cause

Frost heave occurs in certain frost susceptible soil types, such as silts, and requires sustained sub-freezing temperatures and ready access to moisture to occur. Most frost heaves occur in localized areas where a water source is located. As shown in Figure 12.13, frost heave occurs when ice lenses form in soil, which continues to attract water and expand further. The expansion arises from water freezing in the pores of the subgrade soil. Following an initial expansion of approximately 9 percent, water is drawn through capillary action to the areas with the newly formed ice crystals. This results in the continued growth of expanding ice lenses, thereby dramatically increasing the potential for heave. Differential frost heave can occur if there is nonuniformity of the subgrade soils and frost susceptible pockets are present. Although the heaving itself is a problem, the subsequent thawing and differential settlement of the concrete slab can even be a larger problem as it leads to settlement and cracking. Frost susceptible soils such as silts have a low plastic index, meaning a low percentage of water can change the soil from a plastic condition to a liquid condition and cause measurable damage.

Figure 12.13 Mechanisms of frost heave



Jerod Gross, Snyder & Associates, Inc.

Prevention

A number of options are available to reduce the level of differential frost heave, including chemical stabilization of the soil or providing a drainage cutoff system, which intercepts the water source. For example, a typical water source could be a sand lense coming out of a hillside that could be intercepted by a drainage system or when water accumulates behind the back of curbs because of inadequate backfill practices resulting in water source to the frost susceptible soil. Every attempt should be made to determine the water source before any other solutions are considered. This could include soil boring adjacent to the frost heave, if necessary. Removal and replacement of frost susceptible soils would be a last resort due to the cost and availability of acceptable soil.

Expansive Subgrade Soils

Expansive subgrade soils are those that undergo a volume change (swell and shrinkage) in response to increasing and decreasing moisture content in the soil. These changes in volume are more commonly associated with heavy clay materials with high plastic indexes and liquid limits, along with low shrinkage limits (AASHTO A-5 and A-7-5). Although there are numerous clay soils that have the capacity for volume change, montmorillonite has the highest swell potential. The individual crystals of montmorillonite clay are not tightly bound, meaning that water can intervene, causing the clay to swell. The water content of montmorillonite is variable and it increases greatly in volume when it absorbs water. As was the case with frost heave, differential movement can result in localized roughness and cracking in the pavement.

Cause

In order for expansion to occur, the soil must have demonstrated swell potential (generally correlated to soil type) and be exposed to fluctuating moisture contents. The soil index properties listed in Table 12.4 can be used to help determine overall capacity for volume changes in the soils. Higher moisture contents are responsible for the potential expansion and heaves, while excessive drying can result in settlements due to consolidation.

Table 12.4 Characteristics of expansive soils: Relation of soil index properties and probable volume changes for highly plastic soils

Data from Index Tests – Colloid Content (percent minus 0.001 mm) (ASTM D422)	Data from Index Tests – Plasticity Index (ASTM 4318)	Data from Index Tests – Shrinkage limit, percent (ASTM D427)	Estimation of Probable Expansion, percent total volume change (dry to saturated condition)	Degree of Expansion
>28	>35	<11	>30	Very High
20–31	25–41	7–12	20–30	High

Selected parts extracted from Bureau of Reclamation 1998, which cites original source as Holtz 1959, Bureau of Reclamation

Prevention

Prevention measures for potential soil expansion are very similar to those suggested for frost heave. The overall goal is to limit the amount of differential expansion (or contraction) in the soil. This can be accomplished by identifying expansive soils prior to construction, and either removing or replacing them to a sufficient depth such that overburden pressure limits movement or the moisture content remains stable. In rural areas, replacing the unsuitable soils is a common practice. However, in urban areas, replacement soils are sometimes hard to find and are cost prohibitive. In these instances, chemical stabilization becomes effective because of cost and scheduling considerations.

Depending on the soil type, gradation, and mineralogy, the most effective chemical stabilizer is with portland cement, commonly referred to as cement modified soils (CMS). Lime and fly ash have also been used but may not be as effective. Chemical stabilization reduces the soil's cohesiveness (plasticity), improves the overall engineering properties of the soil including reduced moisture sensitivity, increases the strength (level of pavement support k value), reduces shrinkage and expansive characteristics of plastic soils, and provides a uniform level of support. Small quantities of cement (2 to 4 percent) bind soil particles together to form small conglomerate masses of new soil. In addition to the cementing reaction, the surface chemistry of clay particles is improved by the cation exchange phenomena. The causes, prevention, and mitigation techniques are summarized in Table 12.5.

Table 12.5 Summary of causes and prevention or mitigation of settlements and heaves

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Settlements	Inadequate compaction and poor consolidation of support layers Volumetric changes in the subgrade soil due to moisture Excessive moisture in the subgrade	Soil types should be characterized based on mineralogy and engineering properties Compaction specification should be based on optimum moisture and density Control with acceptable stiffness values (CBR, k values, M_r, and/or density) Adjustments may be required as previously discussed (typically wet of optimum) Provide drainage to minimize moisture variations Consider the use of chemical stabilization versus the cost of removal and replacement of poorly graded soils, excessive fine soils, or other problematic soils	Limit the amount of fines (minus No. 200) present in the granular base course to 10% or less to promote free drainage Identify and minimize the use of collapsible soils, frost susceptible soils, or any other soil types that undergo volume changes in response to moisture variations	Proper compaction based on Proctor density requirements Control with acceptable stiffness values (CBR, k values, M_r, and/or density) Blend materials thoroughly prior to compaction to minimize nonuniform support Assure proper gradation control of the granular base course materials Proper installation of a well-designed drainage system	Minimize the intrusion of water in the pavement by routine joint sealing Provide a drainage system and/or roadway ditches to minimize the intrusion of water into the pavement layers Periodic maintenance of the drainage system, including cleanout of the edge drain outlets

Distress in Concrete Pavement	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Heaves	Frost susceptible soils in conjunction with excessive moisture in the subgrade Manholes and in-pavement structures that have stiffer subgrade/base support than the adjacent concrete slab	Soil types should be characterized based on mineralogy and engineering properties Provide drainage to minimize moisture variations Consider the use of chemical stabilization versus the cost of removal and replacement of problematic soils Design the overall pavement structure to account for the local depth of frost penetration	Identify and limit the use of frost-susceptible materials or those that have demonstrated expansion characteristics Limit the amount of fines (minus No. 200) present in the base course to 10% or less to promote free drainage	Proper compaction based on Proctor density requirements Blend materials thoroughly prior to compaction to minimize nonuniform support Assure proper gradation control of the granular base course materials Proper installation of a well-designed drainage system	Minimize the intrusion of water in the pavement by routine joint sealing Provide a drainage system and/or roadway ditches to keep the water table below the frost zone Periodic maintenance of the drainage system, including cleanout of the edge drain outlets

6. Treatment and Repairs

General repair methods and maintenance approaches to address settlements and heaves are described in this section.

Repairs

Full-depth repairs (including slab replacement)

Full-depth repairs can be used to replace cracked/shattered slabs resulting from heaves and settlements. Unlike conventional full-depth repairs, the subgrade and base materials should be removed and replaced, chemically modified, or re-compacted to address the cause of the problem. It should be noted that settlements and heaves may involve multiple slabs or a series of isolated slabs. In either case, the subgrade, base issues, and drainage need to be addressed.

Retrofit Drainage

Settlements and heaves are typically moisture dependent and by limiting the amount of moisture available in the soil, all but the most serious problems can be either resolved or minimized. The use of fin or edge drains or other types of retrofit drainage systems may be applicable but must be considered on a project-by-project basis. A primary purpose of the drainage system in this instance is to intercept the water migrating from sand lenses or other sources to the frost susceptible soil.

Diamond Grinding

In the case of minor settlements and heaves, diamond grinding may be employed to reduce localized roughness. When used in conjunction with slab jacking (or slab stabilization), diamond grinding can provide a relatively long-term solution to localized roughness.

Slab Jacking

Slab jacking can be used to raise localized settlements in pavements.

Maintaining Pavement Subject to Settlements and Heaves

Joint and Crack Sealing

In order to minimize the effects of moisture induced distress, the joints and cracks should be maintained by application of a suitable joint sealant.

Maintain Existing Subdrains and Ditches

Surface water intrusion into the subgrade can be minimized by routine inspection and maintenance of drainage pipes/outlets and ditches.

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CHAPTER 13. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT (CRCP)

1. Description

Continuously reinforced concrete pavements (CRCPs) are generally used for heavily trafficked roadway applications. These pavements differ from the more widely used jointed plain concrete pavements (JPCPs) due to the presence of continuous longitudinal reinforcement, which ranges from approximately 0.70 to 0.80 percent of the cross-sectional area of the pavement slab. Transverse reinforcement is also typically used, which consists of individual bars placed approximately 3 feet apart. Figure 13.1 illustrates typical longitudinal and transverse reinforcement of CRCP (Plei and Tayabji 2012).

Figure 13.1 Concrete reinforcement of CRCP

Plei and Tayabji 2012

CRCP is designed without sawed transverse contraction joints, and is expected to develop hairline random cracks at anywhere from 2 to 8 feet (0.6 to 2.4 m) spacing. Figure 13.2 illustrates a typical transverse crack pattern in a 25-year-old CRCP. The continuous steel reinforcement is intended to keep the cracks from opening, thereby providing adequate load transfer through aggregate interlock. The transverse crack width is generally less than 0.02 inches wide at the top of the slab for well-performing CRCP. Transverse expansion and isolation joints are provided where necessary to prevent compressive stress build-up in the CRCP that result in blowups, or also to isolate the pavement from adjacent structures.

Figure 13.2 Closely spaced transverse cracks in 25-year-old CRCP

Plei and Tayabji 2012

Properly designed and constructed CRCP should experience a limited amount of distress over its design life. However, due to fatigue damage, material degradation, environmental effects, and other factors, rehabilitation will be required at some point. Functional distresses are similar to other types of concrete pavements, and include loss of surface texture, roughness, and others as described elsewhere in this manual. A number of structural distresses are also shared with JPCP, including material-related distresses (MRDs), spalling, and others.

Only distresses specific to CRCP are covered in this chapter. Figure 13.3 shows the various types of CRCP-specific distresses including certain types of longitudinal cracking, irregular transverse cracking, and punchouts (Miller and Bellinger 2014). These will be discussed relative to their causes, severity levels, and preventative measures in this chapter.

Figure 13.3 Specific CRCP distresses

a. High severity transverse cracking in CRCP
Ruiz et al. 2005

b. Moderate severity irregular transverse cracking in CRCP
Global Pavement Consultants, Inc.

c. High severity longitudinal cracking in CRCP
Global Pavement Consultants, Inc.

d. High severity punchout in CRCP
Global Pavement Consultants, Inc.

2. Severity

Pavement distresses specific to CRCP include certain types of longitudinal cracking, irregular transverse cracking, and punchouts. CRCP-specific distresses modified from the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014) and are shown in Table 13.1, as well as a summary of distress levels and measurement criteria.

Determining the cause of, and the most feasible rehabilitation option for, the CRCP distresses indicated in Table 13.1 will require evaluation of the existing pavement support conditions, concrete properties, and a review of the design assumptions, along with the as-constructed pavement properties.

Table 13.1 Summary of CRCP-specific distresses and measurement

Distress	Description	Severity Levels	Measurement
Longitudinal Cracking	Longitudinal cracks are approximately parallel to the centerline of the roadway. These cracks can occur anywhere within the travel lanes.	**Low**: crack widths less than 1/8 in. (3 mm) with no spalling and no faulting **Moderate**: crack widths greater than or equal to 1/8 in. (3 mm) and less than 1/2 in. (13 mm) or spalling less than 3 in. (75 mm) or faulting up to 1/2 in. (13 mm) **High**: crack widths greater than or equal to 1/2 in. (13 mm) or spalling greater than or equal to 3 in. (75 mm) or faulting greater than or equal to 1/2 in. (13 mm)	Longitudinal crack widths and spalling is measured and reported in terms of lineal feet (meters) of each severity level of cracking. Faulting should be recorded at various points along the length of the crack. The integrity of the sealant, if present, should also be recorded as this may be used in determining the appropriate remediation measure.
Irregular Transverse Cracking	Transverse cracks are those that are approximately perpendicular to the centerline of the roadway. Transverse cracks are a normal occurrence in CRCP and only those showing signs of distress (primarily spalling) are of concern.	The number of cracks for each level of severity is recorded, as is the total number of cracks for each predetermined survey section. **Low**: spalling less than or equal to 10% of the crack length **Moderate**: spalling greater than 10% and less than or equal to 50% of the crack length **High**: spalling greater than 50% of the crack length	The lineal feet of spalls should be determined for each transverse crack. The overall crack length and percentage of spalls should also be determined.

Distress	Description	Severity Levels	Measurement
Punchouts	Punchouts are generally regarded as the classic form of CRCP distress. A punchout occurs when closely spaced transverse cracks, or a crack and joint, are intersected by a longitudinal crack relatively close to the pavement edge.	**Low**: longitudinal and transverse cracks remain tight (minimal width) spalling less than 3 in. (75 mm) faulting less than 1/4 in. (6 mm) with no loss of material and no patching **Moderate**: spalling greater than or equal to 3 in. (75 mm) and less than 6 in. (150 mm) or faulting greater than or equal to 1/4 in. (6 mm) and less than 1/2 in. (13 mm) **High**: spalling greater than or equal to 6 in. (150 mm) or faulting greater than or equal to 1/2 in. (13 mm) or concrete in punchout area is loose and moves under traffic, or concrete in punchout area is broken into two or more pieces	Record the number of punchouts for each severity level. The cracks that outline the punchout are also included in the measurements listed above for transverse and longitudinal cracking. Punchouts that have been repaired by completely removing all broken pieces and replacing them with patching material (rigid or flexible) should be rated as a patch.

Source: Modified from Miller and Bellinger 2014

3. Testing

Testing may be both destructive and/or nondestructive depending on the size and relative importance of the project. The overall goals of the testing program are to assess the cause(s) of the distress and the extent of the specific distress type in order to develop viable repair strategies. Refer to Chapter 19 for common laboratory and field testing protocols.

4. Identification of Causes

The most common physical and material/chemical causes of CRCP-specific distresses are shown in Table 13.2. Additional details regarding specific causes are included in the Evaluation section (below). Rehabilitation options generally entail removing and replacing the affected pavement slab(s), as well as remediation of the pavement support layers. Selection of the most cost-effective rehabilitation option requires identifying the most likely cause of the distress.

Table 13.2 Summary of causes of CRCP distresses

Distress	Category	Description
Longitudinal Cracking	Physical	Poor or nonuniform support conditions Variability in subgrade support Inadequate compaction of the subgrade or base during construction Erosion of the base or subgrade Settlement or heaving of the base and/or subgrade Lack of adequate drainage Insufficient depth of cover of the longitudinal steel Late or shallow longitudinal saw cuts Poor consolidation Delayed or inadequate curing Improper longitudinal joint layout
Longitudinal Cracking	Material/Chemical	Poor or nonuniform support conditions Poor quality granular base material Subgrade soils exhibiting a high degree of moisture sensitivity Inadequate subgrade or base stabilization, if applicable
Irregular Transverse Cracking	Physical	Insufficient longitudinal steel content Poor consolidation, particularly around the embedded steel Low or variable concrete strength
Irregular Transverse Cracking	Material/Chemical	High level of friction between slab and base
Punchouts	Physical	Inadequate thickness, resulting in load-induced cracking Poor or nonuniform support Lack of adequate drainage
Punchouts	Material/Chemical	Low or variable concrete strength

5. Evaluation

This evaluation section presents detailed information regarding the most likely causes of the listed distresses and prevention measures. Note that many of the listed factors are applicable to more than one type of distress and that a combination of causes is very likely (Gulden 2013).

Cracking in CRCP can be due to a variety of factors including nonuniform pavement support, poor construction quality, and design-related issues, as shown in Table 13.2. Longitudinal cracks may be load-induced in combination with poor support conditions, particularly if they are in close proximity to the pavement edge. However, those that occur relatively close to a sawed longitudinal joint are more likely to be a function of late or shallow sawing. Differential support can result in both longitudinal and transverse cracking anywhere within the pavement and can be load-induced or due solely to soil or base movement.

Punchouts in CRCP are also due to a variety of factors and require the formation of both longitudinal and transverse cracks. This type of distress requires high repetitions of heavy loads and localized poor support conditions. The progression of a punchout begins with closely spaced transverse cracks bounded by the pavement edge and a longitudinal crack. The area bounded by these discontinuities eventually faults or punches into the weak support layers. The longitudinal steel either yields or ruptures as a result of the concrete displacement, thereby disrupting the continuity of the steel and leading to continued distress development.

Longitudinal Cracking

Cause

Poor or Nonuniform Support Conditions: The long-term performance of CRCP is dependent on adequate and uniform pavement support. This is achieved through proper preparation of the subgrade soil and placement of either an unstabilized aggregate base or a stabilized aggregate base, such as a cement treated base (CTB) or an asphalt treated base (ATB). Failure to achieve uniformity and targeted design values for strength and deformation will typically lead to premature failure. The strength and deformation characteristics of the subgrade and unstabilized bases are based on achieving a target density as determined by laboratory testing prior to construction. The strength of stabilized bases is generally based on unconfined compressive strength for portland cement, fly ash, or lime-treated materials and dynamic modulus for asphalt stabilized materials. The benefit of a stabilized base includes not only strength and uniformity of support but it also serves as a barrier to surface water intrusion and as a construction platform.

Insufficient Depth of Cover: The depth of steel is generally established by the height of the support chairs relative to the slab thickness. Although practices vary somewhat by agency requirements, the depth of steel is typically at mid-depth of the slab or slightly higher. High steel, as it is commonly termed, refers to the steel placement very near the surface, generally 2 inches (50 mm) or less. The lack of concrete cover often results in cracking and spalling directly above the bars. See Figure 13.4.

Figure 13.4 Insufficient depth of cover

Global Pavement Consultants, Inc.

Late or Shallow Longitudinal Saw Cuts: Saw cutting of the longitudinal joints must take place within a specific timeframe, termed the saw cutting window. The window is dependent on ambient conditions, concrete mix characteristics, type of base course (interfacial friction), and overall pavement geometry. In addition, the saw cut must be of sufficient depth to form a plane of weakness in the concrete such that drying shrinkage and thermal stresses will cause a crack to develop under the saw cut. Either late sawing, too shallow of a saw cut, or both can result in development of a random longitudinal crack. See Figure 13.5.

Figure 13.5 Shallow saw cut

Michael Ayers, Global Pavement Consultants, Inc.

Poor Consolidation, Particularly Around the Embedded Steel: Consolidation of the concrete is accomplished through internal vibration. When slipform paving equipment is used for placement (common for CRCP construction), the vibrator array on the slipform paver is responsible for fluidizing and consolidating the concrete. The spacing, amplitude, frequency, and orientation of the vibrators are very important in promoting uniform consolidation of the concrete through the entire slab depth. CRCP has the added requirement of consolidation around the embedded steel such that adequate bond between the concrete and steel is established. Failure to adequately consolidate the concrete will result in excessive entrapped air, lack of bond between the embedded steel and concrete, and many times, lower strength concrete with variable properties. Longitudinal and irregular transverse cracking and punchouts may also be the result of poor consolidation.

Delayed or Inadequate Curing: As it is with all other concrete pavement types, adequate and timely curing is necessary to promote strength development in the concrete, minimize moisture and temperature differentials within the slab during early age strength development, and minimize plastic shrinkage cracking. The curing compound must be applied as soon as possible after finishing and texturing operations are completed. Factors that affect the timing of curing operations include concrete mix characteristics (time of set, rate of strength gain), wind, relative humidity, and temperature, which may vary substantially during a single day's paving. Note that while the use of a membrane curing compound is the most widely used technique, methods including fogging, wet burlap, and plastic sheeting may also be employed.

Improper Longitudinal Joint Layout: Multiple CRCP lanes are typically placed adjacent to one another and "tied" together to prevent lane migration or opening of the longitudinal sawed or construction joints. Depending on the base/slab friction or restraint, tying more than 3 to 4 lanes together may result in the development of random longitudinal cracks, generally in one of the interior lanes.

Prevention

Poor or Nonuniform Support Conditions: Prevention of the listed causes is generally fairly easy to accomplish with a well-executed quality control (QC) plan that's kept in place during construction. Prevention measures for poor or nonuniform support can be found in Chapter 12 of this manual.

Steel Placement: The slab thickness (machine set up) and height of steel must be verified prior to concrete placement. In addition, an adequate number of chairs must be provided to prevent sagging and distortion of the reinforcing steel. The bar spacing and splice laps must also be checked at this time and corrective measures taken, if required.

Consolidation: Consolidation of CRCP is controlled in large part by the vibrator set-up and control when using a slipform paver (Figure 13.6) for placement. The vibrator frequency is most often varied and must be matched to the concrete mix characteristics, slab thickness, paver speed, and the depth and spacing of the embedded steel. Verification of adequate consolidation around the steel should be done during placement of the initial test strip. While no reference test exists to determine the actual level of consolidation, coring and examination of the uniformity of the concrete can be used for QC purposes. Over-vibration can result in segregation of the concrete, a reduction in the entrapped/entrained air content and the likelihood of vibrator trails. Under-vibration may result in lack of consolidation around the steel and nonuniformity of the concrete. It should be noted that a properly engineered concrete mix is generally a more critical element than the paver setup (Taylor et al. 2007).

Figure 13.6 Vibration to fluidize and consolidate concrete

Taylor et al. 2007

Concrete Curing: Curing is very important to prevent the formation of plastic shrinkage cracks, minimize random crack development by helping to control drying shrinkage, and for the concrete to achieve its desired properties. The minimum application rate specified in the contract documents should be verified at the time of application. As a rule of thumb, white pigmented curing compounds should be applied at a rate such that the surface of the slab is uniformly white. The curing compound should be applied as soon as the bleed water evaporates from the surface of the concrete and the finishing and texturing operations are completed. If the curing compound cannot be applied in a timely manner, the use of an evaporation retarder is highly recommended. Other curing methods may also be employed such as fogging/misting, wet burlap, and plastic sheeting.

Joint Sawing: Saw cutting operations for the longitudinal joints must be done as soon as practical after placement without raveling the concrete adjacent to the joint. While the contractor is ultimately in charge of when to initiate saw cutting operations, the critical stresses in the slab, potential for cracking, and the saw cutting window may be estimated using HIPERPAV III, available as a free download at http://www.hiperpav.com/software/. The depth of the saw cut is equally important and is generally equal to one-third of the slab thickness. The depth should be verified frequently during the saw cutting operation and adjustments made, as required. The saw cutting window and depth of saw cut is somewhat dependent on the type of saw used.

Improper Longitudinal Joint Layout: Pavements exceeding approximately three lanes in width that are tied at the longitudinal joints may develop random longitudinal cracks. The number and size of the tie bars at the longitudinal joints may need to be reduced or, in some cases, partially eliminated depending on the base type on which the CRCP is constructed. Stabilized bases offer more frictional resistance to slab movement and may require fewer tie bars to minimize longitudinal cracking, especially if the concrete has a relatively high coefficient of thermal expansion.

Irregular Transverse Cracking

Cause

Design of CRCP differs in many regards to more conventional JPCP. However, many of the fundamental design assumptions are valid for both. The thickness design is based on anticipated traffic loading (weights, configuration, and number), stiffness of the support, and the concrete properties (flexural strength, elastic modulus, and Poisson's ratio). The longitudinal steel content is considered a design variable since it controls the transverse crack spacing, crack width, and load transfer to a large extent.

Inadequate Steel Content: The transverse steel content is designed to hold random longitudinal cracks tightly together, thereby providing load transfer across the crack. However, longitudinal cracking is not a normal occurrence and the transverse steel is routinely placed on 3-foot (0.9 m), center-to-center spacing. Note that the transverse steel is also frequently used in addition to chairs, to support the longitudinal steel bars. The transverse crack pattern in CRCP is largely controlled by the percentage of cross-sectional longitudinal steel and the friction developed at the slab/base interface. Too low of a steel content can result in longer transverse crack spacing and wider cracks. This undesirable crack spacing reduces load transfer and can result in the development of additional cracks close to the existing cracks. This crack pattern can lead to punchouts.

Low or Variable Concrete Strength: A properly calibrated concrete plant, good aggregate stockpile management, and the ability to make adjustments to the concrete as a function of ambient conditions are important factors in maintaining uniform properties. The QC program should incorporate adequate testing and process control measures to comply with the project specifications.

Excess Restraint Due To Bond with Stabilized Base Layer (if applicable): If a stabilized base is used, particularly a CTB, the frictional restraint at the slab/base may be large enough to result in an undesirable, closely spaced crack pattern. The resulting closely spaced cracks are more prone to larger deflections, and higher erosion potential leading to a loss of support and development of punchouts.

Prevention

Steel Content: The longitudinal steel content is based primarily on slab thickness. Common values for longitudinal steel range from approximately 0.7 to 0.8 percent of the cross-sectional area of the slab although significantly higher values are used in some locations based on historical performance. The longitudinal steel content is designed to control both crack spacing and width. Transverse steel is intended to hold any longitudinal cracks that may develop tightly together and promote aggregate interlock load transfer.

Concrete Uniformity: Concrete must meet the minimum strength requirements specified in the contract documents. The concrete strength is an integral part of the design process, and deviation from the target strength can lead to premature fatigue damage to the pavement leading to random cracking and punchouts. Of equal importance is minimizing the variability in workability and strength that have implications to both placement and long-term pavement performance. The risk of cracking is increased at the transition zones between concrete of significantly different strengths.

Excess Base Restraint: Base restraint has an effect on the crack spacing and must be considered in terms of the slab thickness, longitudinal steel content, and environmental conditions. Stabilized bases generally have a high frictional resistance that may not have been adequately considered in the design. In the case of high strength portland cement stabilized bases, cracks may actually propagate from the base into the slab. In order to avoid potential negative issues, a bond breaker is generally used and may consist of a heavy application of curing compound, an asphalt emulsion, or other physical separation layer.

Punchouts

Cause

CRCP design relates strength to the allowable number of anticipated load repetitions. The load carrying capacity is strongly influenced by relatively small changes in slab thickness. If the slab is designed to be thin, random cracks (both longitudinal and transverse) are likely to develop due to fatigue damage. Eventually, punchouts are also likely to occur, well before the end of the pavement design life. In addition, the cross-sectional area of steel is a function of the slab thickness and will be adversely affected.

2. Prevention

Pavement thickness is directly related to the fatigue resistance of the slab. The estimated traffic loading is a critical factor in determining the required slab thickness. The strength of the concrete is of equal importance and should be set to a realistic value based on economic and performance considerations. Although uniformity of the support is very important, the actual level of support has a relatively minor effect on thickness. The causes, prevention, and mitigation techniques are summarized in Table 13.3.

Table 13.3 Summary of causes and prevention or mitigation of CRCP-specific distresses

Distress in CRCP	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Longitudinal Cracking	Variable subgrade soil types Inadequate compaction of the subgrade or base Erosion of the base or subgrade Settlement or heaves Poor quality granular base material Inadequate soil or base stabilization, if used Insufficient depth of cover of the embedded steel Late or shallow longitudinal saw cuts Poor concrete consolidation Inadequate concrete curing Improper longitudinal joint layout Low or variable concrete strength	Joint layout should avoid excessive width Base design must be considered in the overall pavement design Avoid moisture-sensitive soil or base material, or consider drainage system	The concrete strength must be specified to match the design input value Use applicable concrete durability by controlling concrete mix design Unbound aggregate base materials must be wear resistant, have sufficient soundness, have limited fines content, and be of a suitable gradation	Subgrade soils and base materials must be compacted to the target density The longitudinal and transverse steel must be placed to the correct height and with sufficient support The slab must be placed to the target thickness The concrete must be adequately consolidated The concrete must be adequately cured Longitudinal joints must be sawed as soon as possible after final set The saw cut depth must be maintained (typically one-third slab depth)	Random cracks should be sealed if wide enough to allow moisture intrusion Longitudinal joints should be sealed Ensure that ditches and edge drain outlets are maintained

Distress in CRCP	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Irregular Transverse Cracking	Inadequate steel content Poor concrete consolidation Inadequate concrete curing Low or variable concrete strength Excess restraint due to bond with stabilized base layer (if applicable)	The longitudinal steel content should be based on slab thickness, base type, and environmental conditions	The concrete strength must be specified to match the design input value Use applicable concrete durability by controlling concrete mix design Unbound aggregate base materials must be wear resistant, have sufficient soundness, have limited fines content, and be of a suitable gradation	Subgrade soils and base materials must be compacted to the target density The longitudinal and transverse steel must be placed to the correct height and with sufficient support The slab must be placed to the target thickness The concrete must be adequately consolidated The concrete must be adequately cured Longitudinal joints must be sawed as soon as possible after final set The saw cut depth must be maintained (typically one-third slab depth)	Random cracks should be sealed if wide enough to allow moisture intrusion Longitudinal joints should be sealed Ensure that ditches and edge drain outlets are maintained

Distress in CRCP	Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Punchouts	Inadequate slab thickness	Slab thickness must be based on valid traffic, material, support and environmental conditions If punchouts are a common problem, a widened lane or tied concrete shoulder should be considered	The concrete strength must be specified to match the design input value Use applicable concrete durability by controlling concrete mix design Unbound aggregate base materials must be wear resistant, have sufficient soundness, have limited fines content, and be of a suitable gradation	Subgrade soils and base materials must be compacted to the target density The longitudinal and transverse steel must be placed to the correct height and with sufficient support The slab must be placed to the target thickness The concrete must be adequately consolidated The concrete must be adequately cured Longitudinal joints must be sawed as soon as possible after final set The saw cut depth must be maintained (typically one-third slab depth)	Random cracks should be sealed if wide enough to allow moisture intrusion Longitudinal joints should be sealed Ensure that ditches and edge drain outlets are maintained

6. Treatment and Repairs

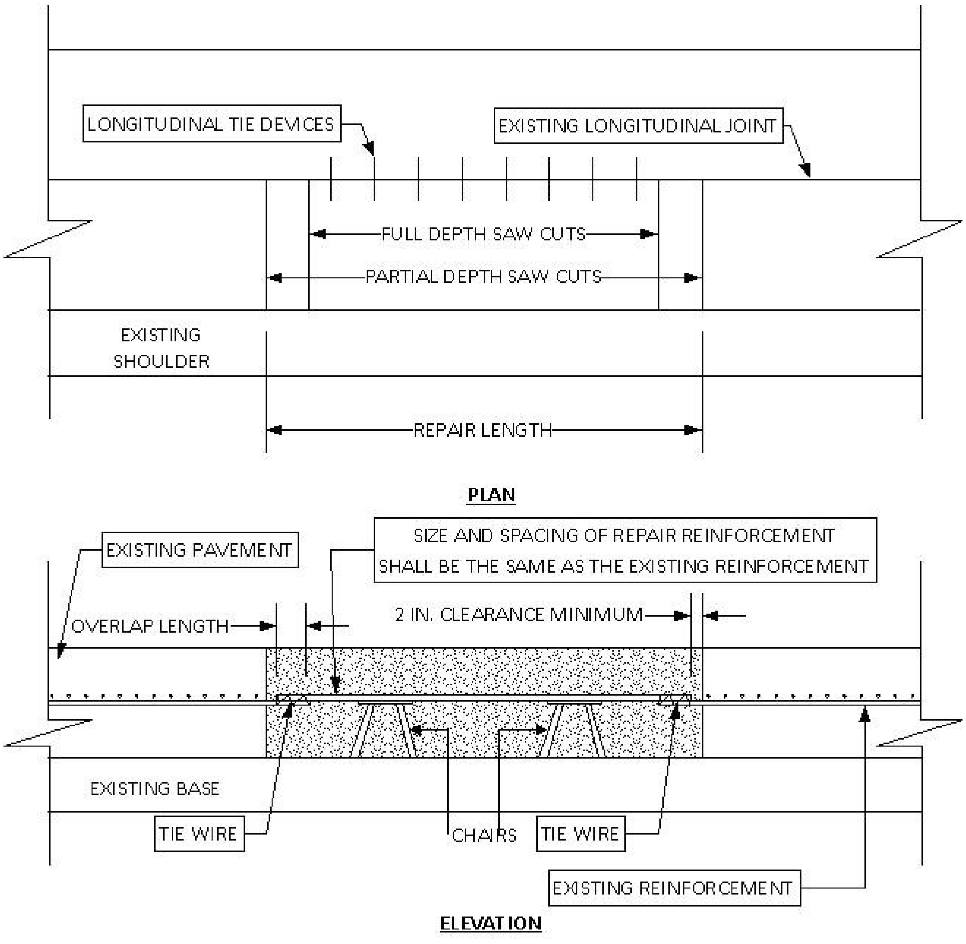
General repair methods and maintenance approaches are described in this section. Many of the causes and prevention measures are applicable to all three distress types (Roesler et al. 2016).

Repairs

Full-Depth Repairs

Full-depth repairs can be used to correct high severity longitudinal and transverse cracks, as well as punchouts at all severity levels. Unlike conventional full-depth repairs, the subgrade and base materials should be removed and replaced in the case of punchouts, and should be considered on a case-by-case basis for crack repair. The support layers may be chemically modified or re-compacted to address the cause of the weakened and variable support. Full-depth repairs in CRCP require maintaining the continuity of the steel, as shown in Figure 13.7.

Figure 13.7 Full-depth repairs



a. Schematic of CRCP full-depth repair

b. Completed full-depth repair on I-77 in South Carolina
Michael Ayers, Global Pavement Consultants Inc.

Retrofit Drainage

Punchouts are typically moisture-dependent and by limiting the amount of moisture available in the subgrade and base, all but the most serious problems can be either resolved or minimized. The use of fin or edge drains or other types of retrofit drainage systems may be applicable but must be considered on a project basis. A primary purpose of the drainage system in this instance is to intercept the water migrating into the pavement from the surface.

Maintaining Pavement

In order to minimize the effects of moisture-induced distress, the joints and cracks should be maintained by application of a suitable joint sealant. If joint deterioration due to the intrusion of deicing chemicals is of concern, it is recommended that the joints be filled their full depth.

Surface water intrusion into the subgrade can be minimized by routine inspection and maintenance of drainage outlets and ditches.

7. References

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CHAPTER 14. INTRODUCTION TO DIVISION 2: CONCRETE OVERLAYS

This chapter will help you to quickly identify where in Division 2 of this manual you can find more detailed guidance on distresses in concrete overlays (causes and solutions). A brief overview of each chapter is provided along with a description of the overlay type being addressed.

When determining the best options for repairing a particular distress, it is important to have a general understanding of the characteristics of the different types of concrete overlays. Although some distresses in concrete overlays are the same as full-depth concrete pavement, there are unique differences. Division 2 addresses the family of bonded and unbonded concrete overlays and distresses that are unique to each overlay type. Concrete overlays are placed over existing asphaltic and concrete pavements that have some degree of distress in them. How the distresses in the underlying pavement are addressed varies for the different overlay types. When addressing distresses that develop in the overlay itself, it is important to understand basic design approaches for each type of overlay. See Table 14.1. Refer to the *Guide to Concrete Overlays* for more detail.

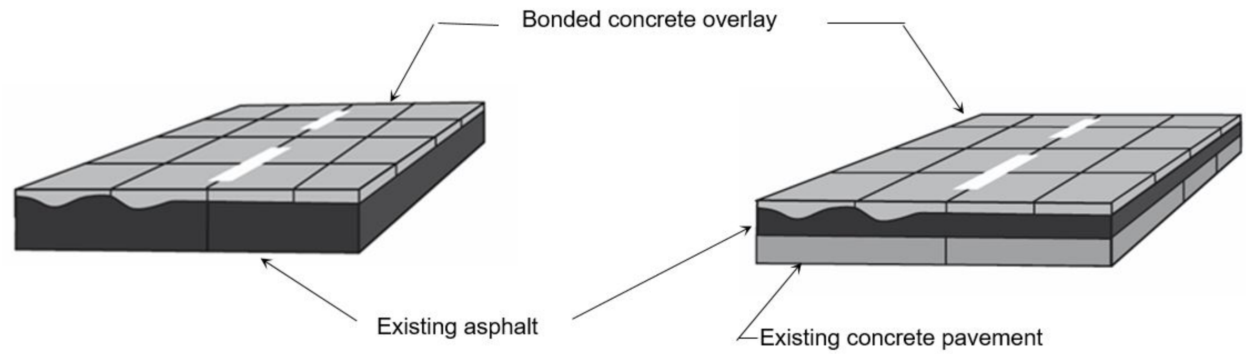
While there are many acronyms used throughout this manual, there are four in particular for Division 2 that are important to be familiar with:

- Bonded concrete overlay on asphalt (BCOA)
- Bonded concrete overlay on concrete (BCOC)
- Unbonded concrete overlay on asphalt (UBCOA)
- Unbonded concrete overlay on concrete (UBCOC)

Table 14.1 Typical design characteristics of bonded and unbonded overlay systems

Overlay Type	Pre-Overlay Pavement Condition	Design Bond Assumption	Pre-Overlay Repairs	Jointing Relative to Underlying Pavement	Typical Overlay Thickness
BCOA and Composite	Good/fair	Bonded	Structurally sound	Not matched	Less than 6 in.
BCOC	Good/fair	Bonded	Structurally sound	Match underlying pavement	Less than 6 in.
UBCOA and Composite	Poor/deteriorated	No bond	Minimal	Not matched	Greater than or equal to 6 in.
UBCOC	Poor/deteriorated	No bond	Minimal	Not matched	Greater than or equal to 6 in.

Chapter 15. Bonded Concrete Overlay on Asphalt (BCOA)

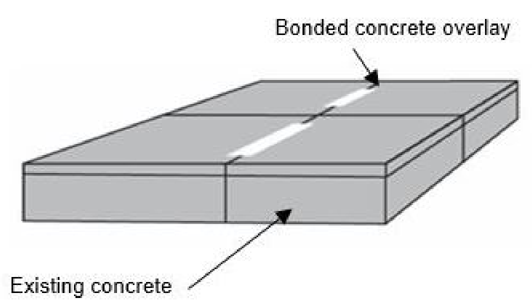


Harrington and Fick 2014, National Concrete Pavement Technology Center

Distress mechanisms that are specific to BCOA include loss of (or failure to develop) adequate bond strength between the concrete overlay and the underlying asphalt, nonuniform support of the concrete overlay panels, and improper sawing of the joints (locations and/or depth). Each of these mechanisms may, in turn, have more than one potential cause (e.g., nonuniform support may result from poor or inadequate pre-overlay repairs, asphalt layer stripping, or other issues).

The resulting BCOA distresses are generally described in conventional terms (i.e., corner breaks or corner cracking, transverse and longitudinal cracking, reflection cracking, wide joints, transverse joint faulting, and joint spalling) but their causes can often be traced to somewhat different mechanisms than for conventional pavement distresses with the same names.

Chapter 16. Bonded Concrete Overlay on Concrete (BCOC)

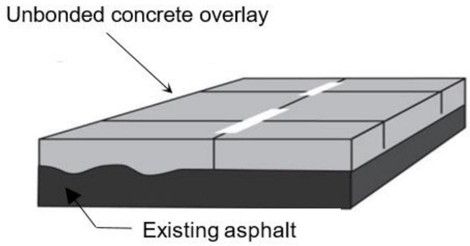


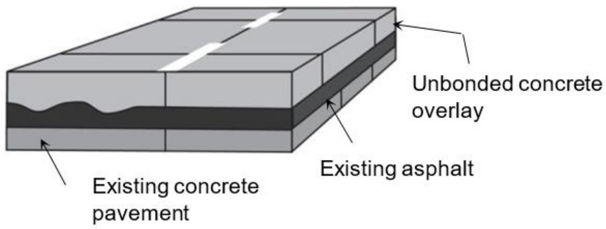
Harrington and Fick 2014, National Concrete Pavement Technology Center

Bonded concrete overlays are relatively thin (typically 2 to 6 inches) concrete layers that are bonded to a pre-existing concrete pavement surface to create a paving layer that acts monolithically. The development and maintenance of the bond between the two layers is directly considered in the overlay thickness design, and is therefore, essential to the performance of the system.

Bonded concrete overlays are also susceptible to a few unique distress mechanisms. For BCOC pavement, these mechanisms are generally related to improper sawing of the joints (locations and/or depth), loss of (or failure to develop) adequate bond strength, inadequate repair of the underlying pavement prior to overlay, and use of the bonded overlay on a poor candidate project. The resulting distresses may initially appear to be conventional cracks or spalls but their causes can be traced to different mechanisms than that of conventional pavement cracking and spalling.

Chapter 17. Unbonded Concrete Overlay on Asphalt (UBCOA)



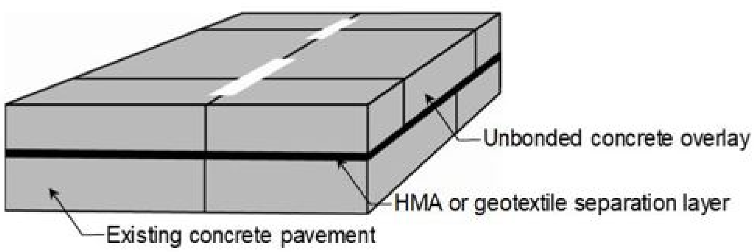


Harrington and Fick 2014, National Concrete Pavement Technology Center

The design of UBCOA treats the existing pavement as a stiff subbase and not as part of the overlay thickness. UBCOA thicknesses have traditionally been somewhat thinner than what would be required by traffic estimates for a full-depth concrete pavement placed on a granular subbase. Typical thickness ranges from 4 to 11 inches. These concrete pavements can be designed as jointed plain concrete pavement (JPCP), with or without load transfer, or continuously reinforced concrete pavement (CRCP). Joint spacing for JPCP unbonded overlays should be a function of the design thickness.

Each of the distresses covered in Chapters 2 through 13 can be also observed in unbonded overlays. However, there are some of these distresses which may manifest themselves differently in UBCOAs. So, for a given distress observed in a UBCOA, the cause(s) may be as described in the appropriate distress chapter for a non-UBCOA pavement, or the cause may be related to the design and construction of the UBCOA.

Chapter 18. Unbonded Concrete Overlay on Concrete (UBCOC)



Harrington and Fick 2014, National Concrete Pavement Technology Center

The design of UBCOCs treats the existing pavement and separation layer as a stiff base, and not as a part of the pavement thickness. Thus, UBCOC thicknesses are only slightly thinner than what would be required by traffic estimates for a full-depth concrete pavement placed on a granular subbase. These concrete pavements can be designed as JPCP, with or without load transfer.

Each of the distresses covered in Chapters 2 through 13 should be consulted for additional information on the identification, causes, evaluation, and treatment of the observed distresses found in UBCOCs. However, there are some of these distresses which may manifest themselves differently in UBCOCs. So, for a given distress observed in a UBCOC, the cause(s) may be as described in the appropriate distress chapter for a non-UBCOC pavement, or the cause may be related to the design and construction of the UBCOC.

Additional Information

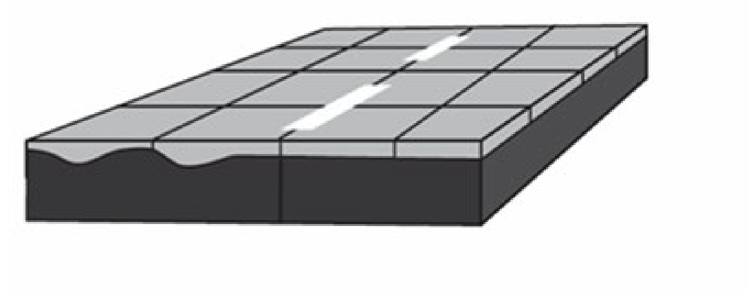
- *Guide Specifications for Concrete Overlays*:
  http://www.cptechcenter.org/technical-library/documents/overlay_guide_specifications.pdf
- *Guide to Concrete Overlays:*
  http://www.cptechcenter.org/technical-library/documents/Overlays_3rd_edition.pdf

CHAPTER 15. BONDED CONCRETE OVERLAY ON ASPHALT (BCOA)

1. Description

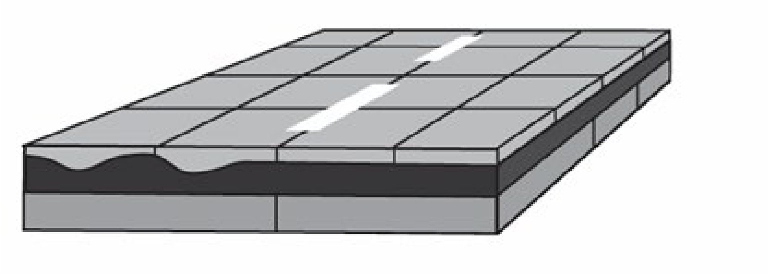
BCOAs are relatively thin (typically 2 to 6 inches) concrete layers that are bonded to a minimum of 3 inches of pre-existing asphalt concrete that is in good, sound condition to create a composite paving layer that acts monolithically. The condition of the existing asphalt pavement is important in order to serve as a monolithic part of the load carrying system with the concrete with both materials performing for the life of the overlay. The development and maintenance of the bond between the two layers is directly considered in the overlay thickness design, and is essential to the performance of the system. See Figure 15.1.

Figure 15.1 Example schematics of bonded concrete overlays on asphalt and asphalt-overlaid concrete pavement



a. Schematic of BCOA

Harrington and Fick 2014, National Concrete Pavement Technology Center



b. Schematic of BCOA of a composite pavement

Harrington and Fick 2014, National Concrete Pavement Technology Center

Because they are constructed using cementitious concrete surfacing, bonded concrete overlays may develop many of the same construction and materials-related distresses commonly observed in conventional jointed or continuous concrete pavements that have been described in Chapters 2–13 of this guide. The bonded pavement system can also develop service-related distresses such as blowups and settlements/heaves by the same mechanisms as conventional concrete pavements. The development, prevention, and treatment of these types of distresses are not repeated in this chapter, which focuses on distresses that are specific to bonded concrete overlay on asphalt (i.e., conventional asphalt pavements and asphalt-overlaid concrete pavements).

There are some distress mechanisms that are specific to BCOA, including but not limited to stripping or other failures within the asphalt layer, loss of (or failure to develop) adequate bond strength between the concrete overlay and the underlying asphalt, nonuniform support of the concrete overlay panels, and improper sawing of the joints (locations and/or depth). Each of these mechanisms may, in turn, have more than one potential cause (e.g., nonuniform support may result from poor or inadequate pre-overlay repairs, localized areas of asphalt layer stripping, or other issues).

The resulting BCOA distresses are generally described in conventional terms (i.e., corner breaks or corner cracking, transverse and longitudinal cracking, reflection cracking, wide joints, transverse joint faulting, and joint spalling) but their causes can often be traced to somewhat different mechanisms than conventional pavement distresses with the same names. Example photos of these BCOA distresses are shown in Figure 15.2. These distresses and their associated causes are the focus of this chapter.

Figure 15.2 Common BCOA-specific distresses

a. Corner cracking at interior locations (early stages)
Tom Burnham, Minnesota DOT

b. Corner cracking at interior locations (late stages), leading to development of shattered panels
Tom Burnham, Minnesota DOT

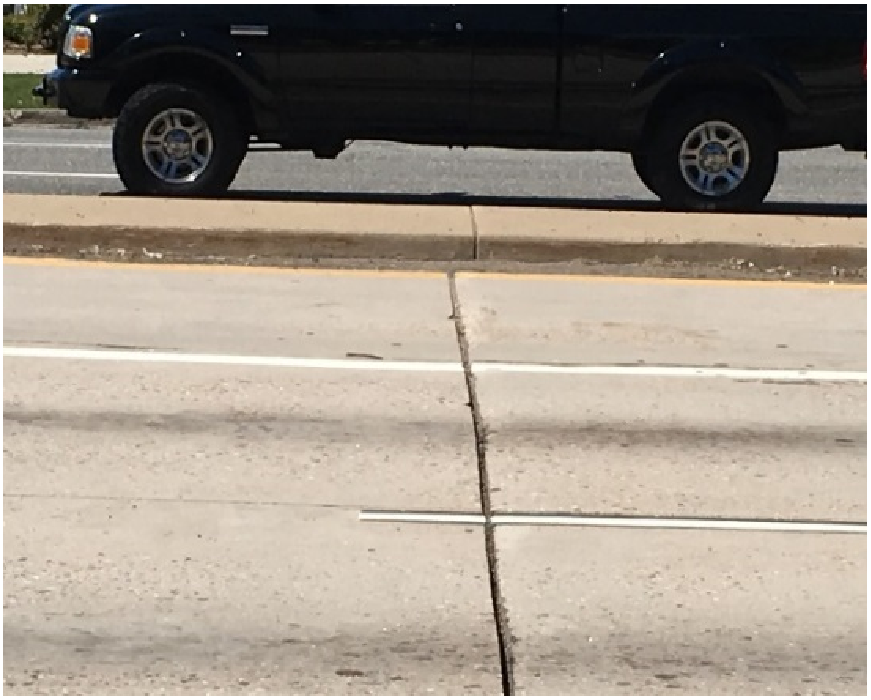
c. Low-severity corner crack at exterior location with evidence of pumping along lane-shoulder joint
Julie Vandenbossche, University of Pittsburgh

d. Well-developed corner cracks at exterior location with evidence of pumping along lane-shoulder joint and secondary transverse cracking

Julie Vandenbossche, University of Pittsburgh

e. Transverse reflection cracking

Julie Vandenbossche, University of Pittsburgh



f. Wide transverse joint and panel migration
Mark Snyder, PERC

g. Panel migration/slippage
Dan DeGraaf, Michigan Concrete Association

h. Longitudinal cracking in wheel path
Julie Vandenbossche, University of Pittsburgh

i. Multiple longitudinal cracking in wheel paths
Mark Snyder, PERC

j. Transverse joint faulting

Tom Burnham, Minnesota DOT

k. Longitudinal lane-shoulder joint spall due to shoulder heave

Kevin Merryman, Iowa DOT

1. Compression at transverse joint due to slab expansion and adjacent joints not opening
Matt Zeller, Concrete Paving Association of Minnesota

2. Severity

BCOA distresses that result from BCOA-specific mechanisms are not addressed in the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014). However, it is reasonable and useful to define these distress severity levels and measurement techniques similarly, as shown in Table 15.1 and concurred on by the Technical Advisory Committee.

Table 15.1 Proposed severity levels and measurement for BCOA-specific distresses

Distress	Description	Severity Levels*	Measurement
Corner Cracking	A full-depth fracture that intersects adjacent transverse and longitudinal joints at an angle of approximately 45 degrees with the direction of traffic. The lengths of the sides are always less than half the width of the slab (by definition) on each side of the corner.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling over less than 10% of crack length, corner fragment is not broken into multiple pieces, and there is no measurable faulting **Medium**: Crack is 1/16 to 1/8-in. (1.5–3.0 mm) wide or spalled (low severity) over more than 10% of length, corner fragment is not broken into multiple pieces, and faulting is less than 1/2 in. (13 mm) **High**: Crack is more than 1/8-in. (3.0 mm) wide, spalled (medium or high severity) over more than 10% of length, corner fragment is broken into two or more pieces and may be loose, or faulting more than or equal to 1/2 in. (13 mm)	Record the number of panels with corner cracks and record the number of corner cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity corner crack and as a patch.
Transverse and Longitudinal Cracking	Transverse cracking is a full-depth fracture that develops roughly perpendicular to the centerline. A longitudinal crack is a full-depth fracture oriented roughly parallel to the pavement centerline or lane-shoulder joint.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling less than 10% of length, adjacent slab fragments are not broken into multiple pieces, and there is no faulting **Medium**: Crack width is 1/16 to 1/8 in. (1.5–3.0 mm) wide, or crack is spalled (low severity) over more than 10% of length, adjacent slab fragments are not broken into multiple pieces, and faulting is less than 3/8 in. (13 mm) **High**: Crack width exceeds 1/8 in. (3mm), or exhibits medium- or high-severity spalling over more than 10% of length, or either adjacent slab fragment is broken into two or more pieces and may be loose, or faulting more than or equal to 3/8 in. (13 mm)	Record the number of panels with transverse and longitudinal cracking and record the number of transverse and longitudinal cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity crack and as a patch.

Distress	Description	Severity Levels*	Measurement
Reflective Cracking*	Reflective cracking is full-depth fracture of the concrete directly above a pre-existing crack in the underlying asphalt pavement.	**Low**: Crack is less than 1/16-in. (1.5mm) wide, spalling over less than 10% of length, adjacent slab fragments are not broken into multiple pieces, and there is no faulting **Medium**: Crack width is 1/16 to 1/8-in. (1.5–3.0 mm) wide, or crack is spalled (low severity) over more than 10% of length, adjacent slab fragments are not broken into multiple pieces, and faulting is less than 3/8 in. (13 mm) **High**: Crack width exceeds 1/8 in. (3mm), or exhibits medium- or high-severity spalling over more than 10% of length, or either adjacent slab fragment is broken into two or more pieces and may be loose, or faulting more than or equal to 3/8 in. (13 mm)	Record the number and length of reflective cracks at each severity level. Rate the total length of the crack at the highest severity level present for at least 10% of the length of the crack.
Wide Transverse Joints/Panel Migration	Wide transverse joints are transverse joints that are significantly wider than originally constructed due to independent longitudinal movement (migration) of the panels.	Not applicable. Severity levels could be defined by categorizing measurements but a complete record of measurements is generally more desirable.	Use an inside-measurement caliper to measure joint width at the top of the joint sealant (or at a distance of 1/4 in. [5 mm] below the pavement surface) in an unspalled area of the joint in the outer wheel path and report to the nearest 1/16 in. [1 mm]. When joint displacement is apparent relative to isolated adjacent structures (e.g., adjacent travel lanes, curb and gutter joints, etc.), measure that displacement and report to the nearest 1/16 in. (1 mm).

Distress	Description	Severity Levels*	Measurement
Transverse Joint Faulting	Faulting is the difference in elevation that develops across a joint after construction.	Not applicable. Severity levels could be defined by categorizing measurements but a complete record of measurements is generally more desirable.	Measure in accordance with Miller and Bellinger (2014), summarized as follows: Use an FHWA-modified Georgia Fault Meter to measure faulting in the outer wheel path and report measurements to the nearest 1 mm.
Longitudinal Lane-Shoulder Joint Spalling (Due to Heave)	Cracking, breaking, chipping, or fraying of the slab edge along the longitudinal joint.	**Low**: Maximum spall width is less than 2 in. (50 mm), when measured to the face of the joint, with no loss of material and no patching, or a spall with no loss of material and no patching **Medium**: Maximum spall width is 2 to 4 in. (50 to 100 mm), when measured to the face of the joint, with loss of material and no patching. **High**: Maximum spall width exceeds 4 in. (100 mm) or spalls containing patch material	Measure in accordance with Miller and Bellinger (2014), summarized as follows: Record the length of longitudinal joint affected at each severity level, recording only spalls that have length of 4 in. (10 cm) or more. Spalls that have been repaired by completely removing all broken pieces and replacing them with patch material should be rated as patches.
Compression Failure at Transverse Joint	Due to slab expansion and adjacent joints not opening	Severity levels can be defined by the relative effect on ride quality and safety.	Record the number of blowups.

*See Evaluation section of this chapter for guidance on distinguishing between conventional and reflection cracking.

Source: Modified from Miller and Bellinger 2014

3. Testing

The mechanisms involved in most distresses that are unique to BCOA include:

- Load-related cracking due to inadequate overlay thickness (e.g., actual traffic being greater than that assumed in design)
- Placement of longitudinal joints in the wheel paths
- Failure to address asphalt thermal cracks in pre-overlay repair and/or location of transverse joints
- Loss of (or failure to develop) bond with the underlying asphalt surface
- Failures in the asphalt layer (e.g., stripping, delamination, etc.)
- Inadequate pre-overlay repairs and/or inappropriate use of BCOA due to poor condition or inadequate thickness of the underlying asphalt-surfaced pavement (i.e., at least 3 inches of sound asphalt, as recommended in Harrington and Fick 2014).

The following sections describe evaluation tools and test techniques for verifying these conditions.

Distress/Condition Survey

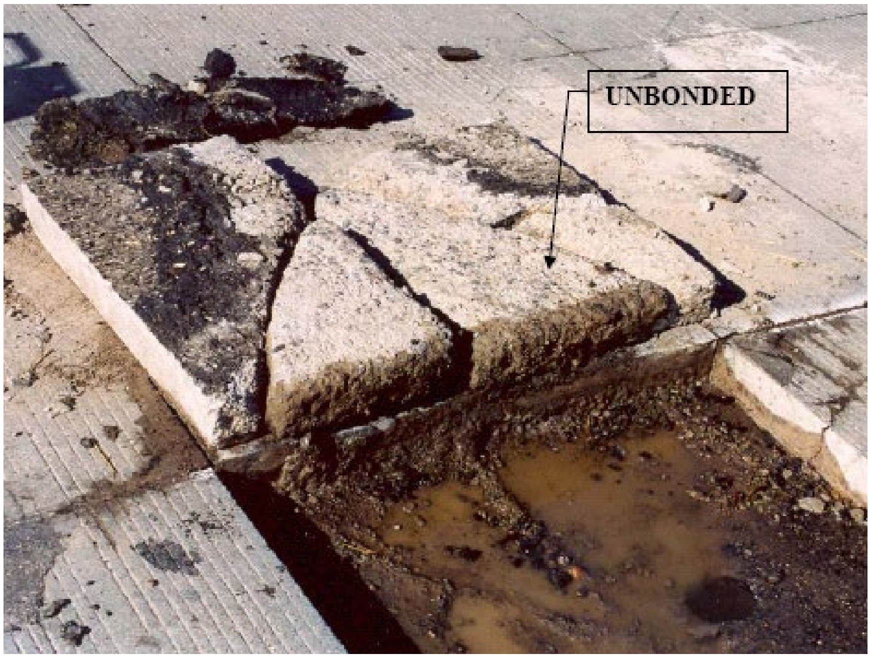
Distress and condition surveys often reveal the probable mechanism of the distress by its form and location (e.g., transverse panel crack aligned with transverse crack in adjacent asphalt shoulder). Field site visits are also an opportunity to verify wheel paths relative to longitudinal joint locations.

Delamination Testing

The presence of debonding/delamination of the two layers can be detected by a wide range of techniques, including:

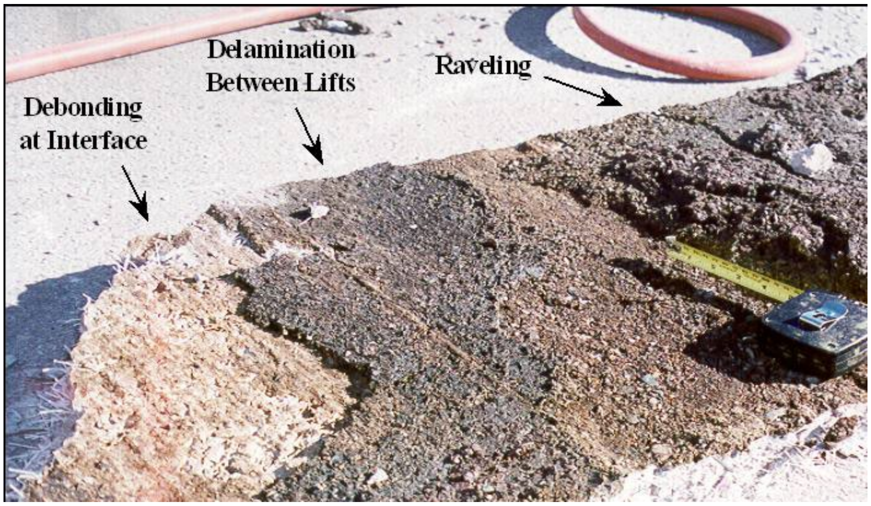
- Direct visual examination of the interface and asphalt material soundness by panel removal (generally as a part of pavement repair; see Figure 15.3);
- "Sounding" of the pavement surface using a chain drag, steel reinforcing bar, small hammer, or other device (sound surfaces produce a relatively clear ringing sound while delaminated surfaces typically produce a dull or "loose" sound; see Smith et al. 2014);
- Coring of suspected debonded areas (care must be taken that the coring operation doesn't produce debonding in the core); and
- Nondestructive testing techniques, such as ground-penetrating radar (GPR) and ultrasonic testing (e.g., MIRA).

Figure 15.3 Examples of direct visual examination of bond interface



a. Example in the field showing a concrete panel that has been removed as an overlay because it has become unbonded

Julie Vandenbossche, University of Pittsburgh



b. Example in the field showing interface debonding, delamination between layers of asphalt, and asphalt raveling—all of which contribute to loss of structural integrity

Julie Vandenbossche, University of Pittsburgh

Coring

Coring is useful in determining actual pavement layer thicknesses in areas of distress for structural evaluation and assessment. Coring may also be useful in assessing the quality of the concrete-asphalt interface bond and in identifying asphalt layer raveling or delamination without removing full panels for investigation (see Figure 15.4). It should be noted that coring processes, in general, can produce bond interface failure within the core, and wet coring processes can produce some raveling or stripping within some asphalt layer materials. In addition, it must be recognized that coring provides an indication of conditions at a single point, which may not be indicative of layer thicknesses and conditions or interface bond conditions everywhere.

For more detailed information on testing, refer to Chapter 19.

Figure 15.4 Asphalt pavement core obtained prior to selecting and designing BCOA

National Concrete Pavement Technology Center

4. Identification of Causes

The causes of most BCOA-specific distresses can generally be traced to issues concerning the suitability of the project for receiving a bonded concrete overlay, certain design and construction issues, and (often to a lesser extent) some material issues. Several of these factors are listed and described briefly in Table 15.2 below. Table 15.3 summarizes the mechanisms of distress for BCOA. More detailed discussions of the contributions of these causes to distress-producing mechanisms are presented in the next section of this chapter.

Table 15.2 Summary of physical and material/chemical causes of distressess in BCOA

Distress	Category	Description of Causes Unique to BCOA
Corner Cracking	Physical	Inadequate slab thickness Load placement (longitudinal joints near wheel path) Loss of bond between portland cement concrete (PCC) and hot mix asphalt (HMA) Inadequate panel edge support (dowel and/or tie bars, aggregate interlock, etc.)

Distress	Category	Description of Causes Unique to BCOA
Corner Cracking	Material/ Chemical	Stripping or raveling of HMA Curl/warp of PCC
Transverse Cracking (Nonreflective)	Physical	Inadequate slab thickness Improper joint layout and resulting panel dimensions Late joint sawing Inadequate HMA thickness or condition Inadequate pre-overlay repair
Longitudinal Cracking (Nonreflective)	Physical	Inadequate slab thickness Improper joint layout and resulting panel dimensions Late joint sawing Inadequate HMA thickness or condition Inadequate pre-overlay repair Loss of bond between PCC and HMA
Longitudinal Cracking (Nonreflective)	Material/ Chemical	Nonuniform overlay support: stripping or raveling of HMA Nonuniform overlay support: curl/warp of PCC
Reflective Cracking	Physical	Asphalt pavement contains a full-depth working crack HMA and PCC are well-bonded and asphalt layer stiffness is greater than that of PCC (considering thickness and layer modulus)
Wide Transverse Joints/Panel Migration	Physical	Seasonal opening/closing of PCC joints Infiltration of incompressibles in poorly sealed/unsealed PCC joints
Transverse Joint Faulting	Physical	Migration of small amounts of asphalt binder and fines under traffic in presence of moisture (stripping), similar to the mechanism for faulting in conventional pavements Shifting of slabs under load on viscoelastic asphalt Inadequate load transfer (dowels or aggregate interlock)
Longitudinal Lane-Shoulder Joint Spalling (Due to Heave)	Physical	Differences in seasonal foundation movements (e.g., swelling soils, frost heave, etc.) between travel lanes and shoulders Inadequate (or nonexistent) ties between travel lane and shoulder pavement

Distress	Category	Description of Causes Unique to BCOA
Compression failure at transverse Joint	Physical	Incompressibles in transverse joints Transverse overlay joints fail to activate properly, resulting in greatly increased effective spacing (and greater seasonal movements) of activated joints Overlay construction under conditions that allow large joint movements and infilling of joints with incompressibles (e.g., construction in extremely warm weather that results in very wide joints in cold weather)

Table 15.3 Summary of mechanisms of BCOA distresses

Item	Description
Inappropriate Application of BCOA	Asphalt is too thin (less than 3 in. of sound asphalt remaining after milling) or is unsound (delaminated or stripped areas) and offers no significant structural contribution to a bonded pavement system
Inadequate Design – Thickness	Overlay design thickness/panel size combination results in high load stresses
Inadequate Design – Joint Layout	Longitudinal joints in wheel paths, panels too large for thickness (curl/warp stress), panels too wide or too long (aspect ratio greater than 1.5)
Inadequate Bond	Inadequate surface prep, stripping/raveling, delamination, ice lensing
Nonuniform Overlay Support	Asphalt too thin, inadequate pre-overlay repair
Insufficient Pre-Overlay Repair	“Soft” or deteriorated areas remain, thermal cracks in stiff asphalt are not repaired, damage done to the remaining asphalt due to construction operation
Inadequate Joint Load Transfer	No mechanical ties or dowels and joints open too wide to rely on aggregate interlock
Unsealed/Unfilled Joints	Water and incompressibles infiltrate freely, accelerating loss of bond, opening of joints and pavement buckling or blowups
Lack of Panel Confinement	Panels can migrate under traffic and expansion/contraction, resulting in increasingly open joints
Consolidation of Asphalt	Viscoelastic asphalt moves (slowly) over time under slab and traffic loads, resulting in small movements of supported panels

Item	Description
Differential Vertical Movement at Lane-Shoulder Joint	Vertical movements of shoulder in response to moisture and/or freezing (e.g., frost heave) and differences in load response across lane-shoulder joint
Construction Conditions Allow Large Joint Movements	Overlay construction under conditions that allow large joint movements and infilling of joints with incompressibles (e.g., construction in extremely warm weather that results in very wide joints in cold weather)
Transverse Joints Fail to Activate	Inadequate saw cut depth and/or environmental and load stresses are not sufficient to cause all transverse joints to activate, resulting in increased effective panel length and greater movements at joints that do activate, allowing easier entrapment of incompressibles

5. Evaluation

The following sections describe the specific causes of each of the previously listed BCOA distresses and discusses approaches for their prevention.

Corner Cracking

Cause

Corner cracking is shown in various locations and levels of severity in Figures 15.2a through d, and is a result of excessive corner stresses due to repeated applied loads acting in combination with one or more additional conditions or mechanisms such as weak support conditions.

Interior corner cracking (as shown in Figures 15.2a and b) is generally associated with the placement of repeated wheel load directly on the interior slab corners, which are the weakest part of each panel (compared to loads placed at a panel edge or a center-of-panel location). Interior corner load placement results from placing a longitudinal joint in one of the wheel paths.

Exterior corner cracking (i.e., cracks that form in the travel lane along the lane-shoulder joint, as shown in Figures 15.2c and d) is a result of the same mechanisms and factors listed above for interior corner cracking, except that the lane-shoulder joint is typically located near but not directly within, a wheel path. In addition, overlay panels placed adjacent to the shoulder may have very little edge support (i.e., no mechanical load transfer across the lane-shoulder joint) when the shoulder is constructed separately and/or of asphalt.

Other factors that can contribute to the development of excessive corner stresses in either interior or exterior locations include inadequate overlay thickness (by design or as-constructed), loss of bond and panel support at the slab corner, and inadequate mechanical load transfer (lack of aggregate interlock) across the longitudinal and transverse joints that form the slab corner. Loss of bond and panel support at the slab corner can be due to slab curling or warping, asphalt stripping or delamination, ice lensing, or other mechanisms.

Prevention

The prevention of corner cracking in BCOA can be accomplished through the approaches discussed below.

Design: Provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using relatively small panels (6 feet by 6 feet or smaller). In addition, take steps in design to ensure adequate support of the panel edges, including the use of dowel load transfer devices and tie bars (when overlay exceeds 7 inches thick and 5 inches thick for tie bars) when the overlay thickness will accommodate them. The two most commonly used BCOA design approaches are mechanistic-empirical (referred to as BCOA-ME) (Li et al. 2016) and short jointed plain concrete pavement (SJPCP/BCOA) in AASHTOWare's Pavement ME Design software (AASHTO 2016).

Materials: Consider the use of structural concrete fibers (especially for overlays that are too thin to easily accept dowel and tie bars) to hold joints tight, thereby controlling panel migration and achieving long-term load transfer through aggregate interlock. Harrington and Fick (2014) provides additional information on the use of structural fibers in concrete overlay mixtures. Also, make sure that the asphalt surface that is to be overlaid is resistant to stripping and raveling and does not contain excessive asphalt binder, which could facilitate overlay slab movement over time.

Construction: Provide a clean asphalt surface to enhance the interface bond. An aggressively textured (e.g., milled) asphalt surface may provide additional shear strength at the bond interface but may not improve overall system performance if the asphalt fails in shear just below the interface (as is sometimes observed). The contractor must control the temperature and moisture of prepared asphalt surface at the time of concrete placement to avoid excessive drying or moisture at the interface that would create a weakened layer of concrete. Use effective curing techniques in a timely manner for the prescribed duration. Avoid the placement of longitudinal joints directly in wheel paths.

Preventive Maintenance: Maintain sealant/filler material in the overlay joints to minimize the infiltration of water, which can collect at the interface. Water collecting at the surface can induce erosion or stripping of the asphalt layer, or cause the formation of ice lenses in freezing climates—all of which reduce the interface bond and increase stresses in the overlay.

Transverse Cracking (Nonreflective)

Cause

Nonreflective transverse cracking is primarily a load-related distress that is relatively uncommon in BCOA with proper thickness design, joint layout, and construction. It may, however, develop in conjunction with advanced corner cracking (see Figure 15.2d) or longitudinal cracking as the effective width of the panel is reduced by the presence of the corner cracking or longitudinal cracks, or in areas of weakened support, loss of bond, or other factors that increase panel deflections and stresses. In addition, transverse cracking may develop early in the life of the overlay (often during the first year of service) due to late joint sawing during construction.

Prevention

The prevention of nonreflective transverse cracking in BCOA can be accomplished through the approaches presented below.

Design: The design considerations described previously for the prevention of corner cracking are also generally applicable to preventing transverse cracking (i.e., provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using relatively small panels (6 feet by 6 feet [2 m x 2 m] or smaller)). In addition, take steps in design to ensure adequate support of the panel edges.

Ensure that the designed joint layout does not result in panels that are too wide. The ratio of width to length should be as close to 1.0 as possible and generally no more than 1.5. Ensure also that the length and width of joints in feet are estimated to 1.5 times the overlay thickness in inches for less than or equal to 6-inch overlays, and 2.0 times for overlays greater than 6 inches. Ensure that longitudinal joints are not within or near the wheel paths. In addition, ensure the proper application of BCOA concepts, including the availability of an adequate thickness of sound asphalt pavement in good condition (i.e., with no raveling or delamination and only minor, if any, fatigue cracking) upon which to place the overlay.

Materials: As described previously for prevention of corner cracking.

Construction: As described previously for prevention of corner cracking, except for the concern about the placement of longitudinal joints in the wheel paths, which has no direct impact on the development of nonreflective transverse cracking. In addition, ensure timely sawing of transverse joints to proper depths.

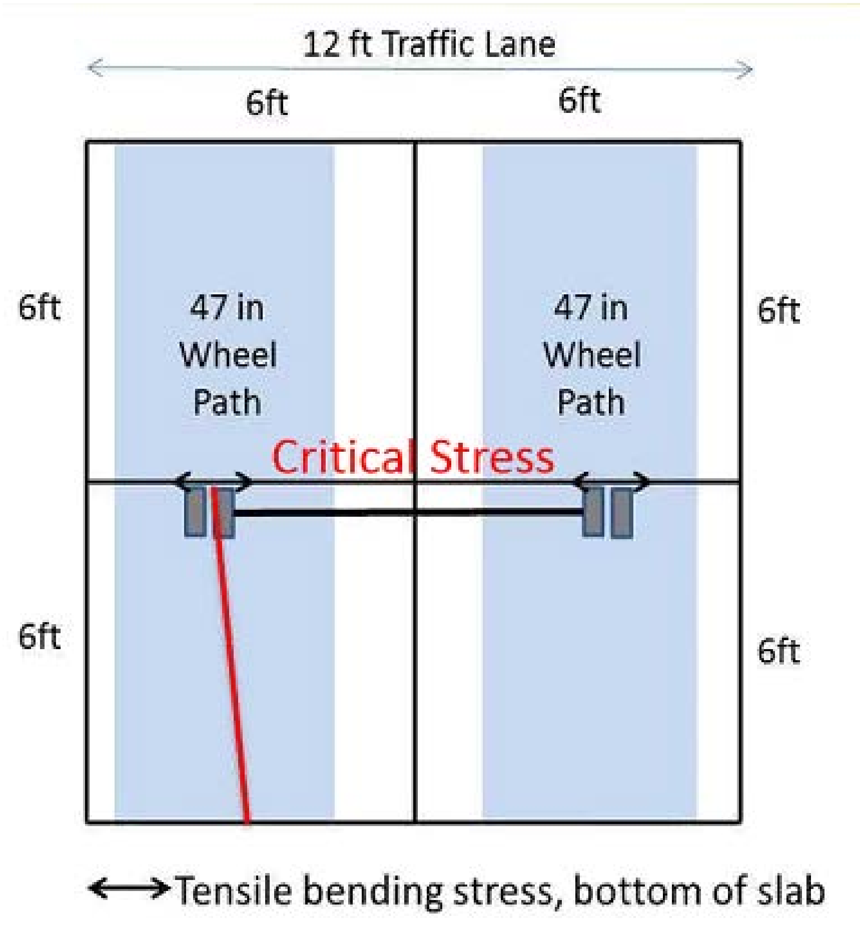
Preventive Maintenance: As described previously for prevention of corner cracking.

Longitudinal Cracking

Cause

As in conventional concrete pavements, longitudinal cracking in BCOA can result from the restraint of early age curl/warp and shrinkage restraint stresses due to the lack of timely sawing of longitudinal joints to the proper saw cut depth. In addition, BCOA longitudinal cracking can result from load-related causes due to inadequate slab thickness, improper panel dimensions and joint layout, loss of bond, nonuniform support of the overlay, inadequate load transfer across transverse joints (leading to high stresses at those joints under passing wheel loads), and/or panel curl/warp stresses. The result of some combination of these factors is longitudinal cracking that generally develops in the wheel paths, as can be seen in Figures 15.2h and i. Note the parallel cracks forming at approximately the same spacing as typical dual wheel spacing in Figure 15.2i. Figure 15.5 further illustrates the manner in which vehicle loads initiate longitudinal cracking at the transverse joints in BCOA.

Figure 15.5 Axle load placement at transverse BCOA joint, resulting in initiation of longitudinal crack (shown in red)



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Prevention

The prevention of longitudinal cracking in BCOA can be accomplished through the following approaches.

As described, ensure that the designed joint layout does not result in:

- Panels that are too wide (a ratio of width to length as close to 1.0 as possible and generally no more than 1.5);
- The length and width of joints in feet more than 1.5 times the overlay thickness in feet for less than or equal to 6-inch overlays, and 2.0 times for overlays greater than 6 inches;
- The placement of longitudinal joints within or near the wheel paths.

Materials: As described previously for prevention of nonreflective transverse cracking.

Construction: As described previously for prevention of corner cracking, except for the concern about the placement of longitudinal joints in the wheel paths, which has no direct impact on the development of longitudinal cracking. In addition, ensure the timely sawing of longitudinal joints to proper depths.

Preventive Maintenance: As described previously for prevention of corner cracking.

Reflective Cracking (Any Type)

Cause

Reflective cracking in BCOA (see Figure 15.2e) most commonly results when the overlay is bonded to an asphalt-surfaced pavement containing a full-depth working crack (typically a thermally induced transverse crack). However, it can also occur with the stiffness of the asphalt pavement layer (considering both layer thickness and elastic modulus) is stiffer than the stiffness of the concrete overlay during a critical season (typically winter, when asphalt is most stiff and pavement joints and cracks open). Details concerning the analytical aspects of this distress mechanism can be found in Vandenbossche 2011.

Prevention

The prevention of reflective cracking in BCOA can be accomplished through the following approaches:

Design: Change the relative stiffnesses of the concrete and asphalt layers (i.e., use thicker concrete overlay and/or remove additional asphalt) such that the concrete layer is always stiffer than the asphalt layer, regardless of seasonal temperature conditions.

Materials: Consider the use of structural concrete fibers and/or mesh or bar steel reinforcing in concrete panels that overlie working transverse cracks in the asphalt layer. Harrington and Fick (2014) provides additional information on the use of structural fibers in concrete overlay mixtures.

Construction: Repair the asphalt crack (full-depth repair) prior to placement of the concrete overlay. Note that the cost of this type of repair may be prohibitive if the cracks are spaced too closely.

Wide Transverse Joints/Panel Migration

Cause

Wide transverse joints and associated panel movement in BCOA (see Figure 15.2f and g) are commonly observed when the joint sealant/filler is not maintained in BCOA joints and the overlay is not confined on all sides (e.g., at an intersection approach where the overlay may extend back a short distance from the intersection, where it meets full-depth asphalt pavement). The transverse joints can become filled with incompressible materials during cool weather when the joints are widest. When the weather warms and the slabs expand, the joints are unable to close so the slabs slide slightly in the direction of least restraint or, if restrained from sliding, may “tent” or blow up (see Figure 15.2l). Over the cycles of several seasons, joint widths can become sufficiently large as to create a noisy, rough ride, and the progressive movement of the panels may amount to several inches, which requires grinding or milling of the asphalt at the boundary of the BCOA.

Prevention

The prevention of wide transverse joints and panel migration in BCOA can be accomplished through the following approaches.

Design: Provide longitudinal joint ties and/or other forms of mechanical restrain to excessive slab movement/migration.

Materials: The use of structural fibers is gaining favor as a technique for providing restraint to joint opening in lieu or (or in addition to) the design-related options above, especially for BCOA with thickness of 4 inches or less, where it is difficult to accommodate tie bars and other structural joint restraints. Harrington and Fick (2014) provides additional information on the use of structural fibers in concrete overlay mixtures.

Maintenance: Maintain joint filler/sealant to minimize the intrusion/collection of incompressibles in joints during cool weather and subsequent slab migration when slabs expand.

Transverse Joint Faulting

Cause

Joint faulting is described as the difference in elevation that develops across a transverse concrete pavement joint over time in response to traffic, materials, and structural and environmental conditions. In BCOA, transverse joint faulting can, in theory, develop through traditional mechanisms (i.e., the pumping-related movement of water and fines under the passage of heavy vehicle loads in the absence of good transverse joint load transfer) if the asphalt-surfaced pavement is cracked full-depth directly below the joint in the concrete overlay. This condition does not generally exist (although it may, occasionally). Significant "faulting" may also develop through settlements of the entire BCOA system (see Figure 15.2).

Since the conventional faulting mechanism is not believed to be generally responsible for the slight development of faulting that has been measured on some BCOA projects, another mechanism must be at work. A few theories have been proposed anecdotally (although none are universally accepted), including:

1. The concrete slabs may shift and move under load on their viscoelastic asphalt foundation, resulting in a slight tipping that mimics faulting.
2. Small amounts of asphalt binder and fines actually do migrate beneath the overlay slabs, under traffic, and in the presence of excess moisture (stripping). They migrate in a similar manner that unbound fines migrate to produce faulting in conventional jointed pavements.

Prevention

Regardless of the mechanism, the following general approaches in design, materials and maintenance seem to offer potential for preventing the development of transverse joint faulting in BCOA:

Design: Provide dowel load transfer systems in overlay slabs that are thick enough to accommodate them (typically 7-inch slabs or greater), and confirm the structural adequacy and stability of the asphalt pavement prior to design and construction.

Materials: Consider the use of structural fiber reinforcement to enhance aggregate interlock load transfer in the overlay. Harrington and Fick (2014) provides additional information on the use of structural fibers in concrete overlay mixtures.

Maintenance: Maintain joint filler/sealant to minimize the intrusion/collection of incompressibles and water in the overlay system.

Longitudinal Lane-Shoulder Joint Spalling (Due to Heaving)

Cause

This distress (shown in Figure 15.2k) is caused by seasonal heaving and differential vertical deflections of the shoulder relative to the concrete overlay resulting in spalling of the overlay (often to the full thickness of the overlay).

Prevention

The following general approaches in design and materials selection seem to offer potential for preventing the development of longitudinal lane-shoulder joint spalling due to shoulder heaving.

Design: Minimize the potential for differential lane-shoulder vertical movements (and subsequent longitudinal joint spalling) through the use of similar structural sections in the mainline and shoulder, lane-shoulder ties and edge drainage.

Materials: Consider the use of frost-resistant shoulder foundation materials.

Compression Failure at Transverse Joint

Cause

This distress develops mainly as a result of the development of high compressive forces in the plane of the pavement, which are typically the result of restraint of slab expansion. This restraint is commonly caused by the infiltration of incompressible materials in wide transverse joints in cooler weather, which prevents the joints from closing properly in warmer weather. Other factors that may lead to wider-than-expected joints in cool weather include overlay construction in very warm weather and the development of “dominant” joints when intermediate joints (adjacent joints) fail to activate soon after construction.

Prevention

The following general approaches in design and materials selection offer potential for preventing the development of compression failure at transverse joints due to the infiltration of incompressible materials in wide transverse joints:

Materials: Consider the use of concrete mixtures with a relatively low coefficient of thermal expansion and reduced potential for shrinkage.

Construction: Ensure that all transverse joints activate soon after construction by using proper saw cut depth. Consider mechanical activation of transverse joints. Seal transverse joints to prevent infiltration of incompressible materials.

Table 15.4 summarizes the most common contributing causes and prevention/mitigation strategies for BCOA-specific distresses.

Table 15.4 Summary of most common contributing causes and prevention or mitigation measures for BCOA-specific distresses

BCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Corner Cracking	Longitudinal joints in wheel paths Inadequate overlay design thickness Inadequate joint load transfer Loss of bond Loss of overlay support (asphalt stripping/raveling, consolidation) Panel curl/warp (minor contribution for small panels)	Provide adequate overlay thickness Use 6-by-6 ft panels or smaller to reduce curl/warp stress Avoid placement of longitudinal joints within wheel paths Use "good" condition asphalt only or restore asphalt to good condition (milling)	Ensure presence of strip- and ravel-resistant asphalt surface material Consider use of structural fiber reinforcement to enhance aggregate interlock load transfer and improve fracture toughness with thickness of less than or equal to 4 in.	Avoid placement of longitudinal joints within wheel paths Provide clean, aggressively textured asphalt surface to enhance interface bond characteristics Control interface temperature and moisture conditions at overlay placement Timely and effective curing of the overlay	Maintain joint filler/sealant to minimize intrusion/collection of water at bond interface (reduce potential for stripping and loss of bond)
Transverse Cracking (Nonreflective)	Inadequate overlay design thickness Loss of bond Effective panel aspect ratio (width to length) greater than 1.5 (panels too long)	Ensure proper application of BCOA concept (adequate thickness of sound asphalt) Provide adequate overlay thickness.	Ensure presence of strip- and ravel-resistant asphalt surface material. Consider use of structural fiber reinforcement to enhance aggregate interlock load transfer	Provide clean, aggressively textured asphalt surface to enhance interface bond characteristics Restore existing asphalt to uniformly sound	Maintain joint filler/sealant to minimize intrusion/collection of water at bond interface (reduce potential for stripping and loss of bond)

BCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
	Effective panel spacing in feet, is no more than 1.5 times the overlay thickness in inches for less than or equal to 6-in. overlays, and 2.0 times for less than 6 in. Panel curl/warp (minor contribution for small panels) Loss of overlay support Inadequate longitudinal joint load transfer Inappropriate use of BCOA/inadequate pre-overlay repairs Late joint sawing	Use 6-by-6 ft panels or smaller to reduce curl/warp stress Use panel aspect ratio less than 1.5 and as close to 1.0 (square) as possible Use "good" condition asphalt only or restore asphalt to good condition (milling)	and improve fracture toughness with thickness of less than or equal to 4 in.	condition (adequate pre-overlay repairs) Timely and effective curing of the overlay Ensure timely sawing of transverse joints to proper depths	
Longitudinal Cracking	Inadequate overlay design thickness Loss of bond Effective panel aspect ratio (width to length) greater than 1.5 (panels too wide) Effective panel spacing in feet, is no more than 1.5	Ensure proper application of BCOA concept (adequate thickness of sound asphalt) Provide adequate overlay thickness.	Ensure presence of strip- and ravel-resistant asphalt surface material. Consider use of structural fiber reinforcement to enhance aggregate interlock load transfer and improve fracture	Provide clean, aggressively textured asphalt surface to enhance interface bond characteristics Restore existing asphalt to uniformly sound condition	Maintain joint filler/sealant to minimize intrusion/collection of water at bond interface (reduce potential for stripping and loss of bond)

BCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
	times the overlay thickness in inches for less than or equal to 6-in. overlays, and 2.0 times for less than 6 in. Panel curl/warp (minor contribution for small panels) Loss of overlay support Inadequate transverse joint load transfer Inappropriate use of BCOA/inadequate pre-overlay repairs Late joint sawing	Use 6x6 panels or smaller to reduce curl/warp stress. Use panel aspect ratio (width to length) less than 1.5 and as close to 1.0 (square) as possible Effective panel spacing in feet, is no more than 1.5 times the overlay thickness in inches for less than or equal to 6-in. overlays, and 2.0 times for less than 6 in.	toughness with "good" condition asphalt only or restore asphalt to good condition (milling)	(adequate pre-overlay repairs) Avoid the placement of longitudinal joints in or near wheel paths Timely and effective curing of the overlay Ensure timely sawing of longitudinal joints to proper depths	
Reflective Cracking	Presence of thermal cracking in asphalt layer Asphalt layer stiffness exceeds concrete layer stiffness (at least seasonally) Inadequate (or nonexistent) repair of asphalt thermal cracking	Increase concrete layer stiffness/thickness Decrease asphalt layer stiffness/thickness	Use fiber or mesh steel reinforcement in concrete over crack.	Repair thermal crack (full-depth repair) prior to overlay placement	N/A

BCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Wide Joints/Panel Migration	Intrusion and accumulation of incompressibles in transverse joints Lack of confinement of panels	Provide longitudinal joint ties or fibers to restraint excessive slab movement and migration	N/A	Keep pavement clean prior to joint sealing	Maintain joint filler/sealant to minimize intrusion/collection of incompressibles in joints during cool weather and subsequent slab movement when slabs expand
Transverse Joint Faulting	Inadequate load transfer across transverse joints Consolidation of asphalt under leave panel Transport of moisture and fines	Provide dowel load transfer greater than 7 in. thick Confirm adequate stability of asphalt layer	Consider use of structural fiber reinforcement to enhance aggregate interlock load transfer	N/A	Maintain joint filler/sealant to minimize intrusion/collection of water and fines
Longitudinal Lane-Shoulder Joint Spalling	Differential vertical movement between shoulder and mainline pavement (often seasonal heaving)	Minimize potential for differential lane-shoulder vertical movement through the use of similar sections, shoulder ties, drainage, and/or frost-resistant shoulder foundation materials	Use frost-resistant shoulder foundation materials.	Exercise care to protect the integrity of the subdrained system	N/A

BCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Compression Failure at Transverse Joint	Incompressibles in transverse joints Increased effective panel length resulting from failure of some transverse joints to activate Overlay construction in extremely warm weather such that joints are open to greater widths in cold weather Concrete mixtures with high shrinkage and CTE	N/A	Use concrete with low CTE and low potential for shrinkage	Saw transverse joints to correct depth Consider mechanical activation of all joints as part of construction process	Maintain joint filler/sealant to minimize intrusion/collection of incompressibles in joints during cool weather and subsequent slab movement when slabs expand

6. Treatment and Repairs

The following treatments and repair methods are each useful for one or more of the BCOA-specific distresses discussed in this chapter. A summary of these general treatment and repair methods and their applicability to address each distress type follows.

Repairs

Partial-Depth Repair

Partial-depth repairs of the BCOA are probably only useful for longitudinal lane-shoulder joint spall repairs, and even then only if the spalls are fairly shallow (i.e., less than half the thickness of the overlay) and localized. The relatively compartmentalized nature of BCOA panels makes the replacement of individual or small groups of panels fairly economical (and, often more reliable repairs than partial-depth repairs of BCOA).

Remove and Replace Individual Panels

The removal and replacement of individual BCOA panels is generally easily accomplished and appropriate for isolated areas of cracking of any type. Existing panel joints can serve as boundaries and forms for rapid repairs. Guidance on performing panel replacements in BCOA can be found in Vandenbossche and Sachs (2013). It should be noted that if the distress is a result of foundation support problems, then subbase/subgrade corrections should be performed to prevent future problems. In these cases, it is common to replace the asphalt and concrete overlay with full-depth concrete panels.

Mill and Inlay Multiple Panels

The removal and replacement of multiple BCOA panels using mill and inlay techniques is a viable option in areas where distress is more widespread. Some adjustment of jointing patterns (i.e., a localized redesign of the overlay) may be possible in the replacement panels to reduce panel stresses and prevent recurrent cracking problems. Vandenbossche and Sachs (2013) provides information on performing this type of repair.

Diamond Grinding

In BCOA, as in conventional jointed concrete paving, diamond grinding is highly effective in removing joint faulting and generally improving concrete pavement ride quality. It should be remembered that faulting will generally re-develop if the mechanisms that caused faulting in the first place are not also addressed at the time of grinding.

Note that the relatively small thickness reduction that accompanies grinding has a greater impact on structural capacity for relatively thin BCOA than for thicker conventional pavements, especially when grinding is performed more than once over the pavement service life. For this reason, it is common to increase the initial thickness of the overlay by 1/2 inch or more to provide a sacrificial layer that can be removed without compromising the intended structural capacity of the overlay.

Maintenance

Rout and Fill/Seal Joints/Cracks

The routing and sealing/filling of panel cracks is not generally recommended. If the cracks are tight and the overlay is not deteriorating, the potential benefits of sealing/filling these cracks is generally considered to be less than the costs (financial and aesthetic) of routing and sealing/filling. If the cracks are wide and deteriorating, panel replacement is often the preferred repair.

Rout and fill/seal may be effective in preventing further widening of joints and panel migration issues where they exist. In addition, studies have shown that there may be potential long-term benefits to preventing the infiltration of water in BCOA joints in order to prevent asphalt stripping and loss of interface bond. This would indicate that BCOA joint resealing programs are potentially beneficial.

Do Nothing

At any given point in time, the best rehabilitation strategy may be to do nothing, particularly when the observed distress is of low severity and is unlikely to significantly and negatively impact pavement performance and ride quality.

Treatment options are summarized in Table 15.5.

Table 15.5 Treatment options for BCOA-specific distresses

Distress	Treatment: Partial-Depth Spall Repair	Treatment: Remove and Replace Panel	Treatment: Mill and Inlay (Multiple Panels)	Treatment: Rout and Seal Cracks/ Joints	Treatment: Diamond Grinding	Do Nothing
Corner Cracking		✓	✓			✓
Transverse Cracking		✓	✓			✓
Longitudinal Cracking		✓	✓			✓
Reflective Cracking		✓		✓		✓
Wide Transverse Joints/Panel Migration			✓ (for extreme cases)	✓ (and mill/grind adjacent shoved pavement, if necessary)		✓ (and mill/ grind adjacent shoved pavement, if necessary)
Transverse Joint Faulting					✓ (only for unacceptable levels)	✓
Longitudinal Lane-Shoulder Joint Spalling (due to Shoulder Heave)	✓	✓ (for large spalls)	✓ (for multiple panels with many larger spalls)			
Tenting or Blowups		✓	✓			

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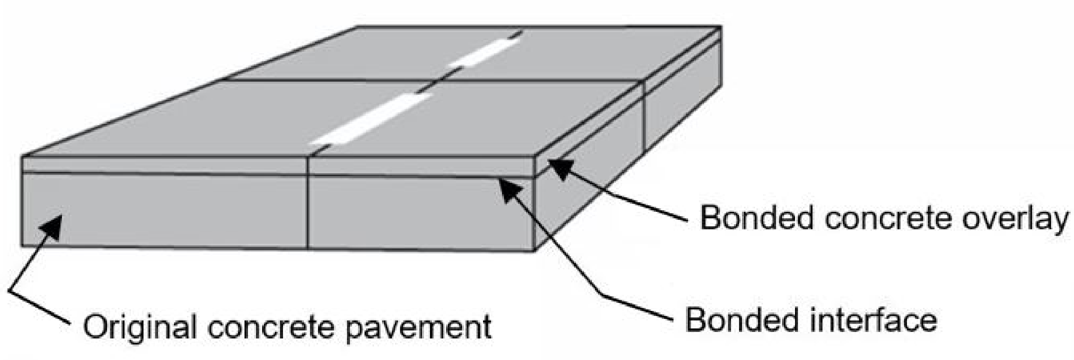
CHAPTER 16. BONDED CONCRETE OVERLAY ON CONCRETE (BCOC)

1. Description

Bonded concrete overlays are relatively thin (typically 2 to 6 inches) concrete layers that are bonded to a pre-existing concrete pavement surface to create a paving layer that acts monolithically (see Figure 16.1). The development and maintenance of bond between the two layers is directly considered in the overlay thickness design and is, therefore, essential to the performance of the system.

Figure 16.1 Illustration of a BCOC



Harrington and Fick 2014, National Concrete Pavement Technology Center

Because they are constructed using cementitious concrete surfacing, bonded concrete overlays may develop many of the same construction- and materials-related distresses commonly observed in conventional jointed or continuous concrete pavements and that have been described in Chapters 2–13 of this guide. These common distresses include various types of surface defects (e.g., map cracking, plastic shrinkage cracking, popouts, etc.) and delamination, joint spalling (due to sawing operations or trapped incompressibles), alkali-aggregate reactivity (i.e., ASR and ACR), D-cracking, and other freeze-thaw-related distresses. The overlaid pavement system can also develop service-related distresses such as panel cracking, corner breaks, joint faulting, blowups, and settlements/heaves by the same mechanisms described in previous chapters for conventional concrete pavements. The development, prevention, and treatment of these types of distresses are not repeated in this chapter.

Bonded concrete overlays are also susceptible to a few unique distress mechanisms. These mechanisms are generally related to improper sawing of the joints (locations, timing, and/or depth), loss of (or failure to develop) adequate bond strength, inadequate repair of the underlying pavement prior to overlay, and use of the bonded overlay on a poor candidate project. The resulting distresses may initially appear to be conventional cracks or spalls but their causes can be traced to different mechanisms than for conventional pavement cracking and spalling. Example photos of these distresses are shown in Figure 16.2. These distresses and their associated causes are the focus of this chapter.

Figure 16.2 Common distresses with mechanisms unique to BCOC

a. Longitudinal crack, possibly due to reflection or joint restraint (improper sawing)
Mark Snyder, PERC

b. Corner crack in overlay due to debonding
Mark Snyder, PERC

c. Reflective crack over unrepaired transverse crack near transition
Mark Snyder, PERC

d. Multiple panel cracks near panel end due to debonding
Mark Snyder, PERC

e. Longitudinal crack due to overlay fatigue after debonding
Randy Riley, Illinois Chapter, Inc. – American Concrete Pavement Association

f. Reflective transverse and longitudinal cracking due to no placement of overlay joints
Randy Riley, Illinois Chapter, Inc. – American Concrete Pavement Association

g. Joint spalling, possibly due to late sawing or saw cut not directly over existing pavement joint/crack

Todd LaTorella, Missouri/Kansas Chapter, American Concrete Pavement Association

h. Longitudinal wheel path cracking due to debonding and fatigue cracking of overlay

Randy Riley, Illinois Chapter, Inc. – American Concrete Pavement Association

2. Severity

Distresses that are unique to BCOC are not specifically addressed in most widely distributed distress identification manuals, such as the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014). However, it is reasonable and useful to define their severity levels and measurement techniques similarly, as proposed in Table 16.1.

Table 16.1 Proposed severity levels and measurement for BCOC-specific distresses

Distress	Description	Severity Levels	Measurement
Corner Cracking (Due to Debonding)	A fracture through the full thickness of the overlay (but not the underlying pavement) that intersects adjacent transverse and longitudinal joints at an angle of approximately 45 degrees with the direction of traffic. The lengths of the sides are always less than half of the width of the slab (by definition) on each side of the corner.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling less than 10% of length; corner fragment is intact **Medium**: Crack exhibits low-severity spalling over more than 10% of length; corner fragment is intact **High**: Crack is spalled (medium or high severity) over more than 10% of length; corner fragment is broken into two or more pieces and may be loose	Record the number of panels with corner cracks and record the number of corner cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity corner crack and as a patch.
Transverse and Longitudinal Cracking (Due to Debonding)	Cracking due to debonding is generally characterized by a fracture through the overlay but not the underlying pavement. Transverse cracking develops roughly perpendicular to the centerline. Longitudinal cracking is oriented roughly parallel to the pavement centerline or lane-shoulder joint.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling less than 10% of length, and adjacent slab fragments are intact **Medium**: Crack width is 1/16 to 1/8-in. (1.5–3.0 mm) wide, or crack is spalled (low severity) over more than 10% of length, and adjacent slab fragments are intact **High**: Crack width exceeds 1/8 in. (3 mm), or exhibits medium or high-severity spalling over more than 10% of length, or either adjacent slab fragment is broken into two or more pieces and may be loose	Record the number of panels with transverse and longitudinal cracking due to debonding and record the number of transverse and longitudinal cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity crack and as a patch.

Distress	Description	Severity Levels	Measurement
Longitudinal Cracking (Joint Compression Failure)	Longitudinal cracking that appears to initiate at the joint (often at a visible inclusion or other source of restraint of joint movement) and propagates in both directions roughly parallel to the pavement centerline or lane-shoulder joint.	Not applicable or defined	Record the number of joints and panels affected and the length of cracking in each panel.
Reflective Cracking (Over Cracks or Joints)	Reflective cracking is a full-depth fracture of the concrete overlay directly above a pre-existing joint or crack in the underlying concrete pavement and where no saw cut was made in the overlay to attempt to match the location of the joint or crack.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling less than 10% of length **Medium**: Crack width is 1/16 to 1/8-in. (1.5–3.0 mm) wide, or crack is spalled (low severity) over more than 10% of length **High**: Crack width exceeds 1/8 in. (3 mm) or exhibits medium- or high-severity spalling over more than 10% of length	Record the number and length of reflective cracks at each severity level. Rate the total length of the crack at the highest severity level present for at least 10% of the length of the crack. Do not double-count with joint spalling due to misplaced saw cut.
Longitudinal Cracking due to Improper Placement of Tie Bars in Widening Unit	Longitudinal cracking due to improper placement of tie bars in a widening unit is characterized by a fracture through the overlay but not the underlying pavement, along a line in the overlay that passes through or near the ends of the tie bars that cross the joint with lane or shoulder widening units.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling over less than 10% of length **Medium**: Crack width is 1/16 to 1/8-in. (1.5–3.0 mm) wide, or crack is spalled (low severity) over more than 10% of length **High**: Crack width exceeds 1/8 in. (3 mm) or exhibits medium- or high-severity spalling over more than 10% of length	Record the number of panels with longitudinal cracking due to improper placement of tie bars in widening units and record the number of longitudinal cracks at each severity level.

Distress	Description	Severity Levels	Measurement
Joint Spalling (Misplaced Saw Cut)	Cracking, breaking, chipping, or fraying of the slab edges between the sawed joint and a reflective crack located within 12 in. (0.3 m) of the sawed joint.	**Low**: Crack adjacent to joint is tight (less than 1/16 in. [1.5 mm]) and the concrete between the crack and the joint is intact **Medium**: Crack adjacent to the joint is 1/16 to 1/8-in. (1.5–3.0 mm) wide and concrete between the crack and the joint is broken into two or more pieces but is still in place **High**: Crack adjacent to the joint is less than 1/8-in. (3 mm) wide or concrete between the crack and joint is broken into two or more pieces or is missing over more than 10% of the length of the joint	Record the number of affected joints at each severity level. Do not double-count with reflective cracking over cracks or joints.
Joint Spalling (Debonding)	Cracking, breaking, chipping, or fraying of slab edges within 12 in. (0.3 m) from the face of any sawed joint with no evidence of reflection cracking.	**Low**: Spalls less than 3 in. (75 mm) to the face of the joint with or without loss of material and no patching **Medium**: Spalls 3 to 6 in. (75–150 mm) to the face of the joint with or without loss of material and no patching **High**: Spalls greater than 6 in. (150 mm) to the face of the joint or spalls containing patch material	Record the number of affected joints at each severity level. A joint is affected only if the total length of spalling is greater than or equal to 10% of the joint length. Rate the entire length of the joint at the highest level present for at least 10% of the joint length.

Source: Modified from Miller and Bellinger 2014

3. Testing

The mechanisms involved in most distresses that are unique to BCOC are listed below and are typically related to loss of bond between the overlay and the underlying pavement or improper construction of overlay joints:

- Pavement is a poor candidate for bonded concrete overlay
- Inadequate overlay thickness
- Inadequate pre-overlay repairs
- Large panel sizes
- Improper location and construction of joint saw cuts
- Dimensional incompatibility (differential expansion/contraction of the overlay relative to the existing pavement)
- Improper use or installation of bond material
- Inadequate pre-overlay surface preparation
- Improper placement of tie bars in widening units
- Delays in concrete placement that result in adverse temperature, moisture, or cleanliness of the base concrete when the overlay concrete is placed
- Late application of curing techniques

The following sections describe evaluation tools and test techniques for verifying these conditions and causes.

Distress or Condition Survey

Distress and condition surveys often reveal the probable mechanism of the distress by its form and location (e.g., cracking adjacent to sawed joints). Field site visits are also an opportunity to verify joint saw cut depth relative to overlay thickness in investigations of localized debonding distresses.

Delamination Testing

The presence of debonding/delamination of the two layers can be detected after construction by using a wide range of techniques, including:

- "Sounding" of the pavement surface using a chain drag, steel reinforcing bar, small hammer, or other device (sound surfaces produce a relatively clear ringing sound while delaminated surfaces typically produce a more dull or "loose" sound; see Smith et al. 2014); and
- Nondestructive testing techniques, such as ground-penetrating radar (GPR) and ultrasonic testing (e.g., MIRA).

Coring

Coring is also useful for determining whether observed cracks extend through the full depth of the composite pavement or just through the overlay and whether saw cut depths are sufficiently deep and wide relative to the underlying joint. Alternatively, if the pavement edges can be exposed, the location and depth of overlay joints relative to the existing pavement joints can be verified.

Evaluating Dimensional Compatibility

Dimensional compatibility should be addressed during materials selection (prior to construction) by:

- Comparing the estimated coefficient of thermal expansion (CTE) values of the selected overlay coarse aggregate and that of the base concrete (from construction and mixture design records or from petrographic examination of material samples);
- Comparing the relative proportions of paste and aggregate in the two materials; and
- Comparing the water to cementitious material (w/cm) ratio of the two materials.

After construction, CTE and shrinkage values can be measured directly using concrete beams or cylinders cut from the concrete overlay and existing concrete pavement using equipment and protocols described in AASHTO T 336-11.

Additional information regarding testing can be found in Chapter 19.

4. Identification of Causes

The causes of most BCOC-specific distresses can be traced to issues concerning the suitability of the project for receiving a bonded concrete overlay, certain design and construction issues, and (often to a lesser extent) some material issues. These factors are listed and described briefly in Table 16.2 below. Table 16.3 summarizes the mechanisms of distress for BCOC. More detailed discussions of the contributions of these causes to distress-producing mechanisms are presented in the next section of this chapter.

Table 16.2 Summary of causes of BCOC-specific distresses

Distress	Category	Possible Contributing Causes
Corner, Transverse, and Longitudinal Cracking (Due to Debonding)	Physical	Failure to develop adequate bond between overlay and original concrete pavement due to inadequate surface preparation prior to overlay placement Vehicle loads on debonded panel corner Large effective panel dimensions (as designed or as constructed) Failure to adequately control placement and curing conditions to minimize potential for early age shrinkage
Corner, Transverse, and Longitudinal Cracking (Due to Debonding)	Material/ Chemical	Differential temperature- or moisture-related expansion/contraction of the overlay and original concrete pavement
Longitudinal Cracking (Localized Joint Compression Failure)	Physical	Entrapment of incompressibles in unsealed joints, resulting in localized bearing points when slabs expand

Distress	Category	Possible Contributing Causes
Reflective Cracking (Over Cracks or Joints)	Physical	Placement of overlay without constructing an overlay joint directly over the existing pavement joint or crack Movement of the underlying pavement that is restrained (until cracking) of the overlay
Joint Spalling (Due to Misplaced Saw Cut)	Physical	Overlay material between the overlay saw cut and the actual joint/crack location in the original pavement becomes debonded due to differential movement of the two layers and breaks up under traffic
Joint Spalling (Due to Debonding)	Physical	Localized debonding along panel edges develops due to physical mechanisms described previously for corner cracking Joint spalling results from load-related stresses in areas of debonding Wobbling or misoriented saw blades create additional force on joint side walls on the up-cut side of the blade, causing debonding and spalling

Table 16.3 Summary of BCOC distress mechanisms

Item	Description
Poor Candidate for BCOC	Construction on pavement in poor structural condition or exhibiting materials-related distress (e.g., D-cracking and alkali-aggregate reaction).
Inadequate Overlay Thickness	Thinner overlays are subject to higher shear stresses under load at the bond interface, potentially making them more susceptible to bond failure.
Inadequate Pre-Overlay Repair of Existing Joints	Areas of joint spalling, delaminated concrete, and nonfunctioning load transfer devices must be removed and replaced with sound concrete (through partial- and full-depth repair) and functioning load transfer devices.
Inadequate Pre-Overlay Repair of Cracks	Working cracks must be addressed with doweled full-depth repairs to avoid cracking and spalling of the overlay, although dowel retrofit and tie bar stitching techniques can be used in some cases.
Large Panel Sizes	Shrinkage and curl-warp stresses increase with panel length and increase the potential for overlay debonding at panel corners and edges.
Inadequate Overlay Joint Design and/or Construction (Depth and Width)	BCOC transverse joints must be cut through the full thickness of the overlay and with a greater width than crack beneath the underlying joint to ensure that the overlay doesn't bridge the underlying joint, resulting in overlay debonding and spalling when joints close.

Item	Description
Dimensional Incompatibility of Overlay and Existing Pavement	If the overlay concrete and original concrete pavement expand and contract (due to temperature and moisture effects) at significantly different rates, the resulting stresses at the bond interface may combine with load-related shear stresses to cause debonding.
Bond Material Failure	Improperly prepared or installed bond agents can introduce a weakened layer or smooth interface that debonds easily.
Inadequate Pre-Overlay Surface Preparation	Failure to create a clean, textured surface that is free of excess water at the time of overlay placement can result in failure at the bond interface.
Improper Placement of Tie Bars in Widening Unit	When tie bars that cross pavement widening joints are epoxied or stapled to the surface of the original pavement, they may reinforce the bottom of the overlay and result in longitudinal cracks near the tie bar ends.
Climate Conditions and Construction Practices During Overlay Construction	Bond strength and concrete strength and durability can be adversely affected by rapid changes in air and concrete temperature, relative humidity, wind speed, timeliness and adequacy of curing, time of concrete placement, cleanliness and moisture of the underlying concrete surface, and other factors.
Failure to Accurately Mark Existing Joint Locations or Failure to Accurately Saw Marked Locations	Saw cuts that are not cut directly over the original joints often result in the formation of a crack a short distance from and parallel to the sawed joint, sometimes resulting in deterioration and loss of overlay material between the crack and the sawed joint.

5. Evaluation

The following sections describe the specific causes of each of the previously listed BCOC distresses and discusses approaches for their prevention.

Corner Cracking Due to Debonding

Cause

Corner cracking of BCOC typically occurs only when the overlay is not bonded near the panel corner, and is subsequently subjected to traffic loads. Fully bonded concrete overlays rarely develop structural cracking if the overlay thickness design is adequate (unless the crack reflects or propagates through the overlay from an unrepaired corner crack in the underlying concrete pavement).

There are many possible causes of overlay debonding, including design-related issues, inadequate surface preparation prior to overlay, issues with bonding materials, curling/warping, inadequate saw cut depth, deteriorated concrete in the existing pavement, or other localized issues. These are discussed in detail below.

Overlay Design-Related Issues

Thickness: Inadequate BCOC thickness is rarely the direct cause of BCOC structural failures because when it's properly bonded to the underlying concrete pavement, the overlay and existing pavement act monolithically, and even small increases in overall pavement thickness result in significantly decreased bending stresses at the top and bottom of the pavement. However, bond shear stress increases with distance of the bond interface from the neutral bending axis (generally assumed to be mid-depth of the composite section), so thinner (e.g., 2 inches) BCOC may be subjected to higher stresses at the interface due to bending than thicker (e.g., 5 inches) BCOC.

Long Panel Lengths: It has been reported (anecdotally) that BCOC placed on long panels (e.g., greater than or equal to 20 feet, typically jointed reinforced concrete pavements) is more susceptible to overlay debonding at the panel corners and transverse joints. This has been attributed to the potential for greater stresses at the bond interface due to the influences of panel length on the development of shrinkage and curl/warp stresses. When BCOC is placed on long panels, some states have cut supplemental joints in the overlay between the joints in the underlying pavement to reduce the overlay joint spacing to 15 feet or less. These supplemental joints need only be cut partial-depth (i.e., through one-third the overlay thickness), although some states have cut them through the full overlay thickness. They serve only to reduce peak stresses at the bond interface and do not initiate the formation of cracks or joints in the existing pavement, so there is no need for dowel load transfer devices at supplemental joints. Furthermore, dowel load transfer devices should never be installed in the bonded concrete overlay as they may induce additional stresses at the bond interface, resulting in debonding.

Inadequate Surface Preparation: Steps that may be taken to prepare the surface after the completion of any pre-overlay repairs include creating a roughened surface or air blasting to further clean the surface.

Creation of a roughened surface enhances bonding between the two layers. Procedures include shot blasting, milling, high-pressure water blasting, and sand blasting. If milling is used, shot blasting or high-pressure water blasting should be used to remove any microcracking or fracturing of the exposed aggregate.

Air blasting will further clean the surface and remove excess water just prior to paving. It is critical that the surface of the existing concrete base be saturated, surface-dry (SSD) or drier—no free water—at the time of overlay placement.

Potential problems with these processes include:

- Microcracking and spalling of the joints and cracks by excessively aggressive milling.
- Failure to adequately remove dust, microfractured concrete, road oils, and other potential contaminants and debonding agents from the existing pavement surface prior to paving.
- Introduction of oil or other contaminants to the pavement surface by using air compressors without oil traps during air- or sand-blasting operations.
- Introduction of dust and other debris to the existing pavement surface by haul trucks and construction operations after the completion of surface cleaning and before paving begins.
- Surface left too wet after water-blasting or washing (i.e., free water is visible) immediately ahead of paving operation.
- Drying or setting of any bonding agent prior to the placement of the overlay.

Materials-Related Issues

Temperature and Moisture Incompatibility of Overlay and Existing Pavement Concrete: The concrete overlay mixture materials and mixture proportions should be selected to minimize the potential for major differences in expansion and contraction or shrinkage, which will add to load-related shear stresses at the bond interface. Failure to adequately address these issues may increase the incidence of distresses related to loss of bond.

The CTE of concrete depends largely on the volumes and types of aggregate in the concrete (especially the coarse aggregate). It is important to use overlay concrete aggregates with similar CTE to those used in the original concrete pavement. Differential expansion/contraction between the two materials can produce high shear stresses at the bond interface whenever pavement temperatures are significantly different than at the time of overlay placement (especially for overlays with higher CTE values).

After years of service, the original concrete pavement will likely have reached a near-equilibrium moisture state and will undergo only small changes in length (volume) in response to changes in moisture condition. The newly placed overlay concrete, however, will undergo both early and long-term shrinkage as it dries. Bond stresses due to differential shrinkage between the overlay and original pavement can be minimized by optimizing the paste (water plus cementitious materials) content of the overlay mixture, and using the least feasible amount of mix water to hold potential shrinkage to a minimum. Guidance concerning the development of low-shrinkage concrete mixtures can be found in Kosmatka and Wilson (2016).

Construction-Related Issues

Bond Material: The use of bond material (e.g., epoxy mortar or cement grout) is generally discouraged (unless specifically required for use with a proprietary repair material) because bond materials may form or create a weak interface between the overlay and original pavement. Cement-based grouts typically have relatively high w/cm ratios, making them weaker than either material they are attempting to bond, and any bond material can dry before the overlay is placed, resulting in a smooth interface that debonds easily. When bond materials are required, they must also be applied properly and in a timely manner and must not be allowed to harden, dry, set, “skin over,” or otherwise cure prior to placement of the overlay.

Climate Conditions and Construction Practices During Concrete Placement. The development of bond between the overlay and existing concrete pavement can be affected by many factors at the time of placement. Climate conditions known to affect the development of bond strength at placement and shortly thereafter include rapid changes in air and concrete temperature (especially rapid cooling, e.g., after rain events on hot days), the time of concrete placement (i.e., night, morning, afternoon or evening, as well as time of year), and relative humidity and wind speed. The interaction of these factors and their impact on concrete strength (and, therefore, bond strength) can be predicted using the FHWA’s HIPERPAV software (The Transtec Group).

While many of the preceding factors are beyond agency and contractor control, additional factors affecting the development of bond that are within contractor/agency control include temperature and moisture condition of the existing concrete pavement surface and concrete mixture at the time of placement, and type, timing, and duration of curing techniques. These factors have the potential to affect the development of both concrete bond strength and interfacial bond stresses due to shrinkage and curling/warping.

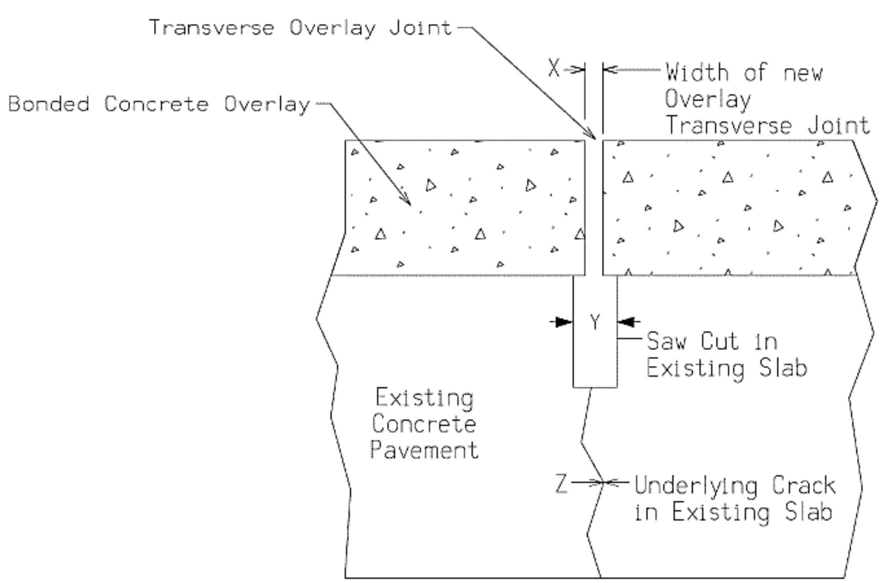
Curing practices are especially important for bonded concrete overlays because the surface area of the pavement is the same as for new construction but the thickness (and volume) of concrete is greatly reduced, so temperature and moisture gradients that develop in the overlay can be much steeper than in conventional pavement construction. High temperature and moisture gradients are associated with higher potential for curl and warp deformations, which put additional stress on the overlay-base concrete interface bond. It is essential that the selected approach to curing is applied as soon as feasible after paving and that it effectively controls both temperature and moisture loss. One common approach in warm weather is to use a double-coat of white-pigmented, wax- or resin-based spray-on curing compound, although water-fogging and plastic sheeting may also be effective. Cure materials should be applied on the exposed vertical surface just as heavily as on the horizontal surface, and should extend at least 1 to 2 inches below the bond interface, if possible.

Inadequate Overlay Joint Depth and Width: Transverse joints in BCOC must be sawed through the full thickness of the overlay (rather than the typical T/3 or T/4 used in conventional paving) to ensure that the overlay does not bridge the underlying joint. Failure to do so may result in all compressive forces in the pavement system (due to thermal expansion, for example) being forced through the "bridge," resulting in spalling and/or debonding of the overlay. Typical design requirements call for cutting the joints to a depth equal to the maximum overlay thickness plus 1/2 inch to ensure that the overlay does not bridge the joints. Longitudinal joints cut over tied joints in the original pavement should be cut to a depth of at least half of the overlay thickness (Harrington and Fick 2014).

BCOC joints must be cut directly above all existing joints and with a saw cut width at least as wide as the widest part of the crack beneath the joint in the underlying pavement at the time of overlay placement; i.e., X must be greater than Y (see Figure 16.3). Overlay joints that are too narrow may result in closure of the overlay joint before the underlying joint closes, which would again force all compressive forces in the pavement system through the overlay, resulting in spalling and/or debonding of the overlay.

If the saw cut (see Figure 16.3) in the existing slab (Y) is 0.5 inches or greater, the underlying crack within the existing slab (Z) should be measured. If Z is 0.25 inches or greater, and existing pavement does not have dowel bars, the joints should be evaluated to determine if load transfer rehabilitation is required to eliminate faulting. If there are numerous joints of this type, the existing pavement may not be a good candidate for BCOC. The existing joints should be filled/sealed to prevent intrusion of mortar during the overlay placement.

Figure 16.3 Illustration of BCOC joint saw cut depth and width requirements



Modified from Harrington and Fick 2014

Prevention

Corner cracking due to debonding can be prevented or mitigated by:

- Improving surface preparation techniques, including using shot blasting and/or water blasting to ensure the removal of all microcracked concrete caused by milling operations, air blasting just prior to concrete placement, and preventing the accumulation of excess water on the interface prior to placement.
- Overlay concrete mixtures must be designed and produced to minimize potential long-term stresses on the overlay-base concrete interface bond, using techniques such as:
    - Overlay aggregate CTE values should be as closely matched as possible to those in the base concrete;
    - Using "optimized" aggregate blends in the overlay concrete, which will permit the use of less paste, reducing potential shrinkage;
    - Use of the lowest feasible w/cm ratio will also reduce overlay shrinkage; and/or
    - Mixture placement temperature should be controlled to closely match or be slightly lower than that of the base concrete.
- Controlling overlay placement and curing conditions to minimize the potential for early age shrinkage.
- Ensuring matched transverse joints are sawed through the full depth of the overlay plus 1/2 inch in a timely manner.
- Reducing stresses on the bond interface by reducing panel sizes in the overlay (by adding intermediate cuts between existing pavement joints).
- Maintaining overlay joint seals, especially in freeze-thaw climates where infiltrated water might otherwise penetrate the bond interface and freeze.

Transverse and Longitudinal Cracking Due to Debonding

Causes

Transverse and longitudinal cracking of BCOC typically occurs only when the overlay is not bonded near the panel edges and is subsequently subjected to traffic loads. Fully bonded concrete overlays of concrete pavement rarely develop transverse and longitudinal structural cracking if the overlay thickness design is adequate—unless the crack reflects (propagates) through the overlay from an unrepaired crack in the underlying concrete pavement.

There are many possible causes of overlay debonding, including design-related issues, inadequate surface preparation prior to overlay, issues with bonding materials, curling/warping, inadequate saw cut depth, deteriorated concrete in the existing pavement, or other localized issues. The causes and mechanisms of BCOC bond failure are discussed in detail in the preceding section on corner cracking, and therefore are not repeated here.

Prevention

The primary mechanism for transverse and longitudinal cracking (due to debonding) is debonding of the overlay at the panel edges due to curling/warping, deteriorated concrete in the existing pavement, or other localized issues, usually in combination with traffic loading. It can be prevented or mitigated by the same techniques listed previously for corner cracking due to debonding.

Longitudinal Cracking (Localized Joint Compression Failure)

Causes

Longitudinal BCOC cracking can develop due to high compressive stresses at concentrated areas of the transverse joints during times of slab expansion (i.e., warm weather). The most common source of concentrated restraint is the intrusion of incompressible materials in unsealed joints. Another potential source is the failure to cut transverse joints through the full depth of the overlay, leaving a small thickness of concrete overlay that bridges the underlying joint.

Prevention

This type of overlay cracking can be prevented by sawing overlay joints through the full overlay thickness, and by constructing and maintaining good overlay joint seals.

Reflective Cracking

Causes

Reflective cracking results when an overlay is placed over existing pavement joints and cracks without constructing an overlay joint directly above the existing pavement joint or crack. If the overlay remains bonded, movements in either the overlay or underlying pavement will result in the development of overlay cracking directly over the joints or cracks in the original pavement.

Prevention

Reflective cracking can be avoided by sawing overlay joints directly above the existing pavement joints or cracks and through the full overlay thickness (see Figures 16.4 and 16.5). If this cannot be done because the path of the existing pavement joint/crack is irregular and difficult to follow, a full-depth repair of the existing pavement can be performed to produce two straight joints that should be duplicated in the overlay.

Figure 16.4 Bonded concrete overlay joint placed directly over full-depth repair joint in original pavement

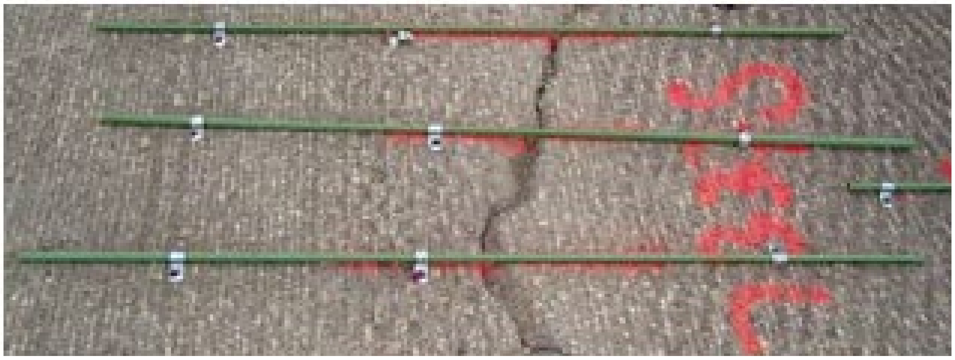
Mark Snyder, PERC

Figure 16.5 Saw cut in bonded concrete overlay over working transverse crack in underlying pavement on I-255 near St. Louis, Missouri

John Donahue, Missouri DOT

Repairs of and joint sawing over the crack do not need to occur when tight, nonworking cracks exist but a tight reflection crack should be expected. Cross-stitching or slot-stitching can also be used prior to the overlay placement to help hold longitudinal cracks tight. Tie bars have also been "stapled" to the surface of the existing pavement over existing longitudinal cracks for the same purpose (see Figure 16.6).

Figure 16.6 Reinforcing steel anchored to concrete pavement over tight crack



James Cable, Iowa State University

Joint Spalling (Misplaced Saw Cut)

Causes

This distress occurs when the overlay joint saw cut does not line up with the joint in the existing pavement, resulting in the formation of a parallel reflective crack in the overlay directly over the joint in the underlying pavement and very close to the overlay saw cut (see Figure 16.7). The concrete between the saw cut and reflective crack often deteriorates under traffic and is ejected (spalling) from between the crack and the saw cut.

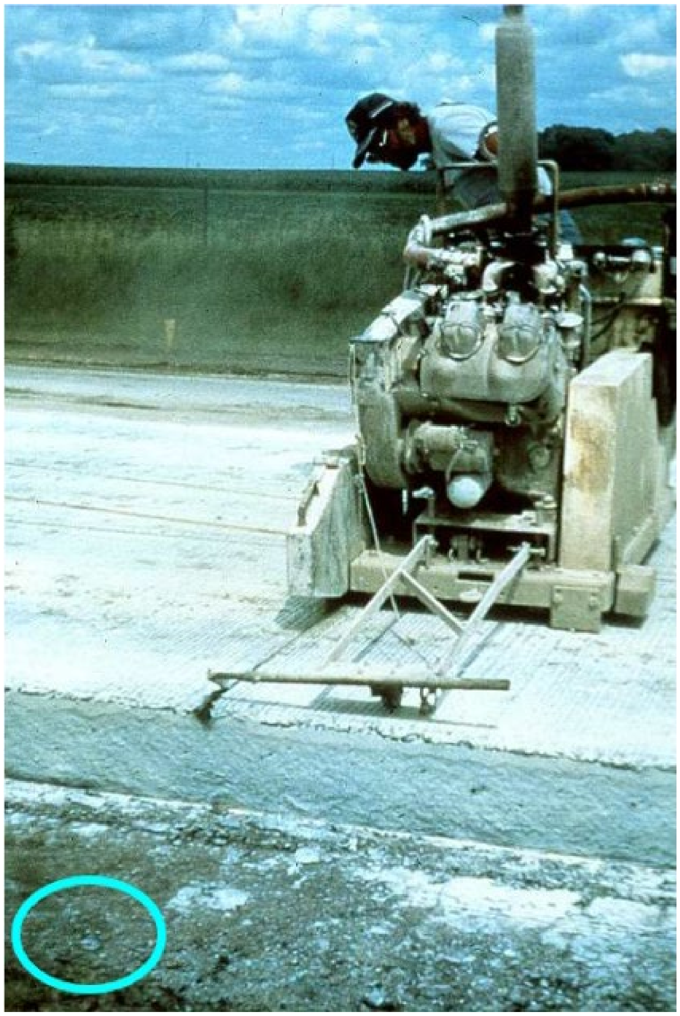
Figure 16.7 Reflective crack due to not cutting a transverse joint over an existing and underlining concrete pavement crack

Todd LaTorella, Missouri/Kansas Chapter, American Concrete Pavement Association

Prevention

This distress can be prevented with improved joint marking and sawing operations. Figure 16.8 shows one technique for marking base concrete joint locations (nail and washer in shoulder) and guiding the saw cut; other options are also available.

Figure 16.8 Use of nail and washer in shoulder (see circled area) to mark existing joint line prior to overlay placement and joint sawing



Randy Riley, Illinois Chapter, Inc. – American Concrete Paving Association

Joint Spalling (Debonding)

Causes

This distress is typically caused by debonding of the overlay along the panel edges, often due to curling/warping and/or inadequate surface preparation or other localized issues, combined with load-related stresses and pavement movements.

Joint sawing equipment and practices can also induce joint spalling in BCOC. If the saw blades bind in the pavement (e.g., due to blade wobble or slight misorientation of the blade relative to the slot), the additional force on the joint side walls on the up-cut side of the blade may cause localized debonding and spalling of the overlay.

Prevention

As with the other debonding-related distresses, joint spalling due to debonding at the joints can be prevented or mitigated by reducing stresses at the bond interface during design and construction, and by improving the quality of the surface preparation during construction to enhance bond quality. These processes are described in detail in the section above that describes corner cracking due to debonding.

In addition, efforts must be made to ensure that sawing operations are performed with well-maintained equipment (i.e., blades are not warped, arbor bearings are properly adjusted and not worn, etc.) and good construction techniques to prevent binding of the saw blades against the sides of the overlay joint.

Table 16.4 summarizes the common causes and prevention/mitigation strategies for the distresses that are unique to bonded concrete overlays of concrete pavement.

Table 16.4 Summary of most common contributing causes and prevention or mitigation measures for BCOC-specific distresses

BCOC Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Corner Cracking (Due to Debonding)	Debonding of the overlay at the corner due to curl/warp, deteriorated existing concrete, or other localized issues	Reduce panel lengths (add intermediate joints)	Minimize potential for differential expansion between overlay and existing concrete (modify mix proportioning, select overlay aggregate with similar CTE to existing pavement aggregate)	Reduce panel length (cut intermediate joints) Enhance surface preparation and cleaning. Improve placement conditions Improved curing	Maintain joint seals, especially in freeze-thaw climates
Transverse and Longitudinal Cracking (Due to Debonding)	Debonding of overlay near panel edges due to curl/warp, deteriorated existing concrete, or other localized issues	Reduce panel lengths (add intermediate joints)	Minimize potential for differential expansion between overlay and existing concrete (modify mix proportioning, select overlay aggregate with similar CTE to existing pavement aggregate)	Reduce panel length (cut intermediate joints) Enhance surface prep and cleaning Improve placement and curing conditions	Maintain joint seals, especially in freeze-thaw climates
Longitudinal Cracking (Joint Compression Failure)	Localized restraint of transverse joint function, generally due to intrusion of incompressibles in the overlay lift	N/A	N/A	Construct and maintain good transverse joint seals	Maintain transverse joint seals or otherwise prevent/remove incompressible materials from transverse joints

BCOC Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Reflective Cracking (Over Cracks or Joints)	Placement of overlay directly over cracks or joints without sawing a joint in the overlay	Design or specify full-depth repair of cracks in existing pavement and saw overlay joints over (and into) all joints in existing pavement	N/A	Perform full-depth repair of cracks in existing pavement and saw overlay joints over (and into) all joints in existing pavement	N/A
Joint Spalling (Misplaced Saw Cut)	Saw cut in overlay does not line up with joint in existing pavement or is not cut through the full thickness of the overlay	N/A	N/A	Improve pre-overlay work to enable accurate location of joints after placement of overlay; improve quality of saw cutting to better track marked joint locations	N/A
Joint Spalling (Due to Debonding)	Debonding of the overlay at the panel edge due to curl/warp, deteriorated existing concrete, or other localized issues	Reduce panel lengths	Consider potential for differential expansion between overlay and existing concrete (mix proportioning, aggregate selection)	Reduce panel length Enhance surface prep and cleaning Improve placement and curing conditions	Maintain joint seals, especially in freeze-thaw climates

6. Treatment and Repairs

General treatment and repair methods to address concrete pavement surface defects are described below and are summarized in Table 16.5.

Repairs

Full-Depth Repair/Panel Removals

Full-depth repairs are generally necessary in areas where distresses are the result of inadequate pre-overlay repair of the underlying pavement (e.g., where full-depth cracking is present, materials-related deterioration of the existing pavement extends to more than one-third to one-half of the original slab thickness, or where existing load transfer systems no longer function properly, etc.). These repairs should generally be constructed using repair design and construction techniques described in Smith et al. (2014).

Partial-Depth Repair/Concrete Inlay

Partial-depth repairs of the bonded overlay may be appropriate in areas where observed distresses are caused by localized areas of delamination that are attributed to inadequate surface preparation prior to placement of the overlay (and the underlying concrete is sound). Concrete inlays (i.e., milling along or just below the original bond line and inlaying a new lift of concrete) may be appropriate for larger areas of delamination that meet the criteria described above for partial-depth repairs. Care must be taken not to cause or extend areas of delamination when milling existing materials for inlay construction.

Maintenance

Rout or Saw (and Seal)

If BCOC distresses are attributed (fully or in part) to inadequate joint saw cut depth or width, it may be possible to prevent additional joint areas from debonding by re-sawing the joints to a more appropriate depth and/or width.

Table 16.5 Treatment options for BCOC-specific distresses

Distress	Treatment: Full-Depth Repair	Treatment: Partial-Depth Repair	Treatment: Mill and Inlay	Treatment: Rout or Re-Saw Joint (and Seal, if applicable)	Do Nothing	Reconstruct
Corner Cracking (Due to Debonding)	✓	✓				
Transverse and Longitudinal Cracking (Due to Debonding)	✓		✓			✓ (For widespread debonding)
Longitudinal Cracking (Joint Compression Failure)				✓ (Consider using epoxy crack injection or HMWM to “glue” crack)		
Reflective Cracking (Over Cracks or Joints)	✓				✓ (May be appropriate for reflective crack of low-severity crack in existing pavement)	
Joint Spalling (Due to Debonding)		✓	✓ (Type II partial-depth repair)			
Joint Spalling (Misplaced Saw Cut)		✓	✓ (Type II partial-depth repair)			

7. References

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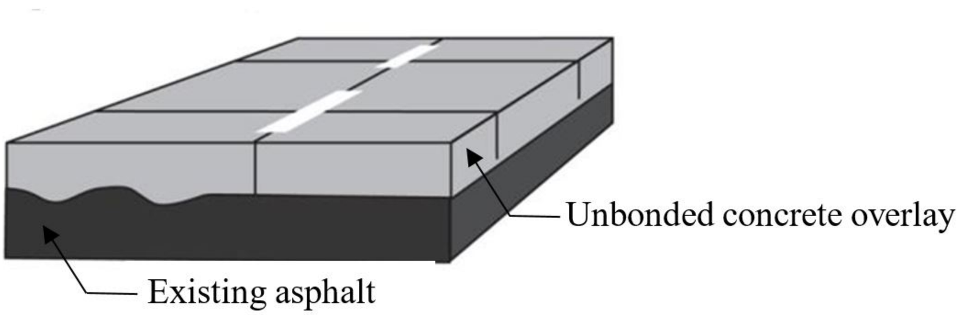
CHAPTER 17. UNBONDED CONCRETE OVERLAY ON ASPHALT (UBCOA)

1. Description

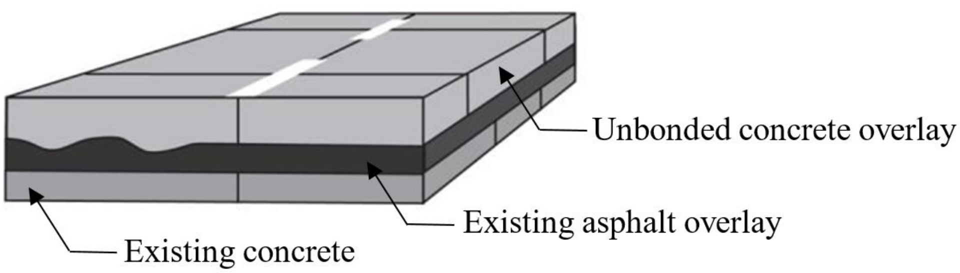
Unbonded concrete overlay on asphalt (UBCOA) can be found on all functional classifications of roadways. By definition, UBCOAs consist of a new portland cement concrete surface placed over an existing asphalt surfaced pavement. This can be a full-depth asphalt pavement or a composite pavement (existing concrete that has been overlaid with asphalt). The existing pavement serves as a subbase for the new concrete surface (Figures 17.1 and 17.2).

Figure 17.1 Unbonded concrete overlay on existing asphalt pavement



Harrington and Fick 2014, National Concrete Pavement Technology Center

Figure 17.2 Unbonded concrete overlay on existing composite pavement



Harrington and Fick 2014, National Concrete Pavement Technology Center

By design, UBCOAs treat the existing pavement as a stiff subbase and not as part of the overlay thickness. This stiff subbase sets the value of the modulus of subgrade reaction (k value), which is used in overlay design thickness calculations. However, the k value does not significantly affect the overlay thickness. Thus, UBCOA thicknesses have not been thinner than what would be required by traffic estimates for a full-depth concrete pavement placed on a granular subbase. Typical thickness ranges from 4 to 11 inches. These concrete pavements can be designed as jointed plain concrete pavement (JPCP), with or without load transfer, or as a continuously reinforced concrete pavement (CRCP). Joint spacing for JPCP unbonded overlays should be a function of the design thickness.

Each of the distresses covered in Chapters 2 through 13 can be observed in unbonded overlays. These chapters should be consulted for additional information on the identification, causes, evaluation, and treatment of the observed distresses found in UBCOAs. However, there are some of these distresses which may manifest themselves differently in UBCOAs. So, for a given distress observed in a UBCOA, the cause(s) may be as described in the appropriate distress chapter for a pavements other than UBCOA; or the cause may be related to the design and construction of the UBCOA. This chapter deals specifically with unique causes of distresses observed in UBCOAs.

Example photos of these UBCOA distresses are shown in Figure 17.3. These distresses and their associated causes are the focus of this chapter.

Figure 17.3 UBCOA-specific distresses

a. Longitudinal cracking in the wheel path of a UBCOA with tied shoulders and widened sections
Kevin Merryman, Iowa DOT

b. Longitudinal cracking in wheel path
Kevin Merryman, Iowa DOT

c. Diagonal longitudinal crack over widened section
Kevin Merryman, Iowa DOT

d. Longitudinal crack over widened section
Kevin Merryman, Iowa DOT

e. Corner cracking

Andy Bennett, Michigan DOT

f. Mid-panel cracking

Tom Burnham, Minnesota DOT

g. Transverse cracking due to severely misaligned dowels; paint marks show the dowel basket as located using GPR

Matt Zeller, Concrete Paving Association of Minnesota

h. Blowups

Matt Zeller, Concrete Paving Association of Minnesota

2. Severity

Table 17.1 summarizes the severity and measurement of distresses which are unique to UBCOAs. The information in the table is derived from the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014).

Table 17.1 Proposed severity levels and measurement for UBCOA-specific distresses¶

Distress	Description	Severity Levels	Extent Levels
Longitudinal Cracking in the Wheel Path of a UBCOA with Tied Shoulders and Widened Sections **Longitudinal Cracking in Wheel Path** **Transverse Cracking**	A longitudinal crack is a full depth fracture of the concrete overlay, orientated roughly parallel to the pavement centerline or the lane - shoulder joint. A transverse crack is a full-depth fracture of the concrete overlay that develops roughly perpendicular to the centerline.	**Low**: Crack is less than 1/16 in. (1.5mm) wide, spalling over less than 10% of length, adjacent slab fragments are not broken into multiple pieces, and there is no faulting **Medium**: Crack width is 1/16 to 1/8 in. (1.5 to 3.0 mm) wide, or crack is spalled (low severity) over more than 10% of length, adjacent slab fragments are not broken into multiple pieces, and faulting is less than 1/2 in. (13 mm) **High**: Crack width exceeds 1/8 in. (3mm), or exhibits medium- or high-severity spalling over greater than 10% of length, or either adjacent slab fragment is broken into two or more pieces and may be loose, or faulting greater than or equal to 1/2 in. (13 mm)	Record the number of panels with transverse and longitudinal cracking and record the number of transverse and longitudinal cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity crack and as a patch.
Corner Cracking	A corner crack is a full-depth fracture of the concrete overlay that intersects adjacent transverse and longitudinal joints at an angle of approximate 45° with the direction of the traffic. The lengths of the sides are always less than one-half the width of the slab on each side of the corner.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling over less than 10% of crack length, corner fragment is not broken into multiple pieces, and there is no measurable faulting **Medium**: Crack is spalled (low severity) over more than 10% of length, corner fragment is not broken into multiple pieces, and faulting is less than 1/2 in. (13 mm) **High**: Crack is spalled (medium or high severity) over more than 10% of length, corner fragment is broken into two or more pieces and may be loose, or faulting is greater than or equal to 1/2 in. (13 mm)	Record the number of panels with corner cracks and record the number of corner cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity corner crack and as a patch.

Distress	Description	Severity Levels	Extent Levels
Transverse Joint Faulting	A difference of elevation across a joint or a crack of the concrete overlay.	**Low**: Less than or equal to 1/8 in. (3mm) fault **Medium**: 1/8 to 3/8 in. (3 to 9.5 mm) fault **High**: Greater than 3/8 in. (9.5 mm) fault	**Occasional**: Faulting occurs along less than 20% of the joints and cracks **Frequent**: Faulting occurs along 20 to 50% of the joints and cracks **Extensive**: Greater than 50% of the joints and cracks are faulted
Blowups	The result of localized upward movement of the concrete overlay or shattering of the overlay along the transverse joint or crack.	N/A	N/A

Source: Modified from Miller and Bellinger 2014

3. Testing

The mechanisms involved in the most common distresses that are unique to UBCOAs are listed below.

- Inadequate evaluation of existing conditions, resulting in deficient design
- Load-related cracking due to insufficient as-built overlay thickness and/or actual traffic being greater than that assumed in design
- Placement of longitudinal joints in the wheel paths
- Loss of support due to poor drainage which results in stripping of the existing hot mix asphalt (HMA) layer(s)
- Nonworking transverse joints
- Misplaced load transfer dowels
- Panel movement due to stripping and/or shear failure of the underlying HMA

These mechanisms can result in one or many of the distresses and may be identified through field surveys and coring where appropriate, and as described below.

Distress/Condition Survey

Distress and condition surveys often reveal the probable mechanism of the distress by its form and location (e.g., transverse panel crack aligned with transverse crack in adjacent asphalt shoulder). Field site visits are also an opportunity to verify wheel paths relative to longitudinal joint locations.

Coring

Coring is useful in determining actual pavement layer thicknesses in areas of distress for structural evaluation and assessment. Coring can also provide valuable information regarding:

- Identification of stripping in the existing HMA layer(s);
- Deterioration of the underlying portland cement concrete pavement in composite sections;
- Determining whether transverse joints have developed (cracked under the saw cut), resulting in working joints; and
- As-built thickness of the UBCOA.

Nondestructive Testing

There are a number of nondestructive methods that may be helpful in locating load transfer dowels and tie bars. Depending upon the distress observed (e.g., cracking near the joint, longitudinal cracking, transverse joint faulting, etc.), locating the embedded steel items may assist in diagnosing the root cause of the distress. Common nondestructive methods available include those listed below.

- Magnetic imaging tomography scanning for dowel bars; note this method may not be suitable for dowels placed in baskets if the shipping wires were left intact, or if the existing HMA contains slag aggregate
- MIT scan T2 for tie bars
- Ground penetrating radar to find the general location of embedded steel
- Pachometer to find the general location of embedded steel

The applicability of these devices to specific distresses is covered in previous chapters. Manufacturer's recommendations should be consulted for proper operation of these devices.

More information regarding testing can be found in Chapter 19.

4. Identification of Causes

The causes of most UBCOA-specific distresses can generally be traced to improper evaluation of the existing conditions, certain design and construction issues, and (often to a lesser extent) some material issues. Several of these factors are listed and described briefly in Table 17.2. More detailed discussions of appropriate distress causes unique to UBCOAs follow Table 17.2.

Table 17.2 Summary of causes of UBCOA-specific distresses

Distress	Category	Description of Causes Unique to UBCOA
Longitudinal Cracking in Tied Shoulders and Widened Sections	Physical	Differential movement (heaving) of materials underlying shoulders and/or widened sections Tie bar placed at the bottom of the UBCOA slab Too many tie bars per slab Inadequate compaction in widened section
Longitudinal Cracking in Tied Shoulders and Widened Sections	Material/Chemical	Tie bar size larger than No. 4
Longitudinal Cracking In Wheel Path	Physical	Inadequate slab thickness Improper slab dimensions or wheel overloading Inadequate base support or Inadequate load transfer Asphalt stripping and/or scouring Inadequate tie bar placement, particularly with tied shoulders Inadequate subbase support under widened sections Excessive curling/warping
Corner Cracking	Physical	Longitudinal joint near wheel path Inadequate slab thickness

Distress	Category	Description of Causes Unique to UBCOA
		Inadequate load transfer Lack of edge support Panel movement
Mid-Panel Cracking	Physical	Loss of support due to poor drainage and subsequent stripping of the existing HMA Inadequate thickness of existing HMA after milling Inadequate overlay thickness and/or wheel overloading Reflective cracking of thermal cracks in the existing HMA
Mid-Panel Cracking	Material/Chemical	Existing HMA susceptible to stripping
Transverse Cracking	Physical	Misaligned dowels due to movement of the dowel basket Inappropriate slab dimensions Structural inadequacy resulting in premature fatigue Differential movement between the UBCOA and underlying HMA during initial set
Transverse Joint Faulting	Physical	Deformation of the underlying asphalt under repeated loading Slab movement Erosion of the asphalt binder and fines underneath the joint Inadequate remaining thickness of HMA leading to inadequate base support Inadequate load transfer Panel movement
Blowups	Physical	Undeployed joints in the UBCOA Panel movement

5. Evaluation

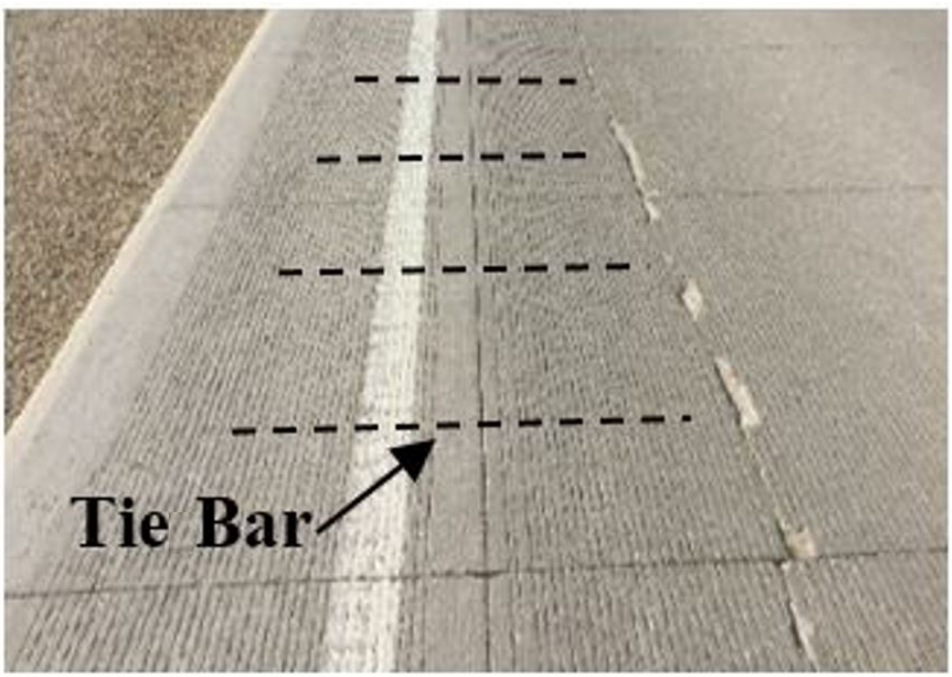
The following sections describe the specific cause of each of the previously listed UBCOA distresses, and briefly discusses approaches for their prevention.

Longitudinal Cracking in Tied Shoulders and Widened Sections

Cause

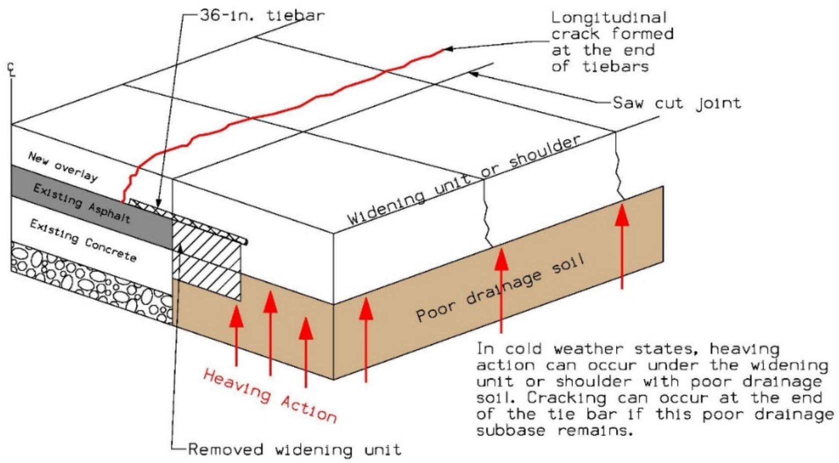
Widening or shoulder heaving: Some UBCOAs have been placed over older, narrow sections of roadway which have either been previously widened or are widened during the construction of the UBCOA. Similar to these widened sections, other UBCOAs have been constructed with tied concrete shoulders or widening units; where the underlying shoulder materials offer less support to the UBCOA than the existing mainline pavement section. In both cases, and in frost-susceptible environments, when heaving occurs in the poorly drained materials underlying the shoulder/widened section, or an excessive load is applied and the joint is not allowed to hinge properly, a longitudinal crack can form at the end of the tie bars. See Figure 17.4. Many of these UBCOA widening units were designed and constructed with a tie bar stapled to the top of the HMA separation layer. See Figure 17.5.

Figure 17.4 Longitudinal cracking from vertical displacement of widening



Kevin Merryman, Iowa DOT

Figure 17.5 Detail showing how inadequate subbase stability and excessive loads under widened sections contributes to diagonal longitudinal cracking in cold weather regions



Snyder & Associates, Inc.

Prevention

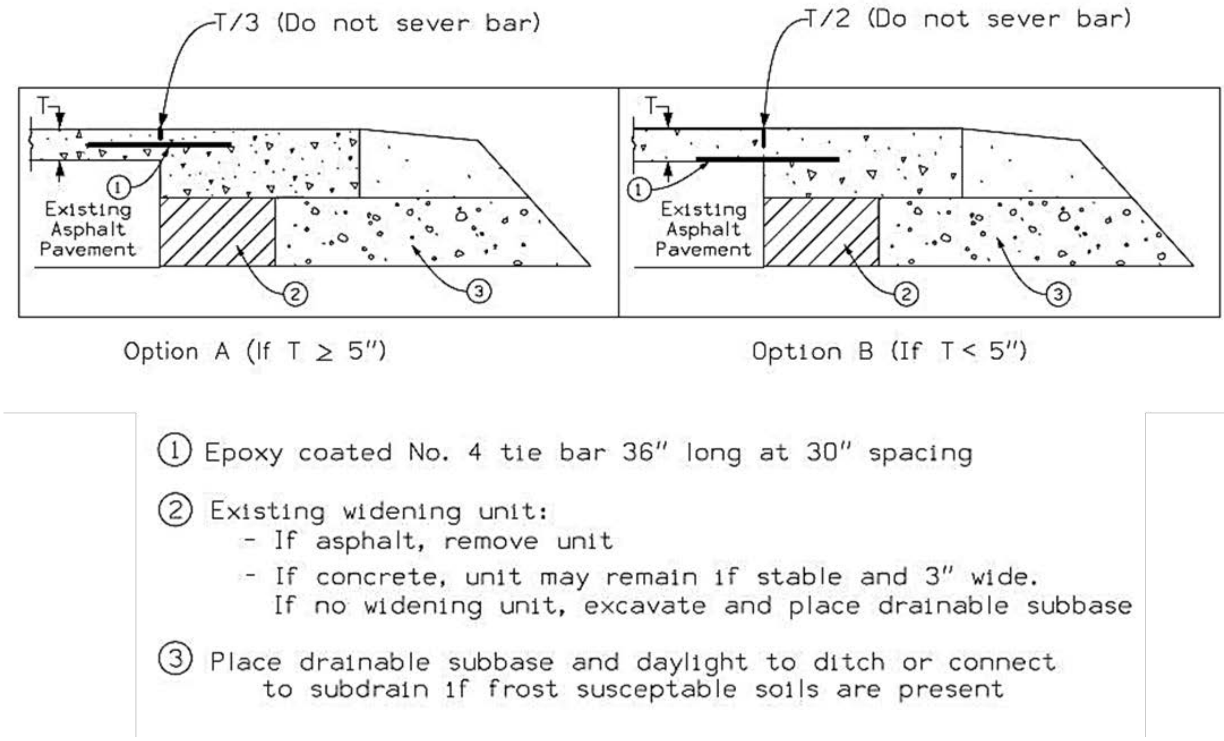
Interim guidance recommends placing the tie bar at the neutral axis (mid-depth) of the UBCOA whenever possible, or alternatively using structural fibers to provide post-cracking integrity of the UBCOA. Also, replacement of a poorly drained soil with a drainable subbase is recommended for projects in frost-susceptible environments.

Design: Provide adequate overlay thickness to allow placement of bars with a minimum of 2-inch cover above and below the tie bars or utilize structural fiber reinforcement. Also, provide adequate drainable subbase support (stable and non-expansive) under the widened section. When the widening unit is not as thick as the combined new overlay and existing pavement thickness, the subbase stability must be taken into consideration. Under heavy truck traffic loading, the granular subbase may need to be stabilized (e.g., cement-treated base).

If the UBCOA is 5 inches or greater in thickness, the tie bars are recommended to be installed at mid-depth of the new overlay centered over the existing edge of pavement. See Figure 17.6 Option A. Saw depth should be T/3 and care should be taken not to cut the tie bar.

If the UBCOA is less than 5 inches thick, the tie bars may be placed at the bottom of the overlay by paver inserter or secured with epoxy or nails. See Figure 17.6 Option B. Saw depth should be T/2. With Option B, the tie bar at the bottom of the overlay does not allow the longitudinal joint at the widening or shoulder section to hinge. Therefore, if this option is selected, it is recommended to remove frost-heave susceptible soil and replace with a drainable subbase in frost-susceptible environments. The drainable subbase should daylight to the ditch or connect to a working subdrain system.

Figure 17.6 Examples of tie bar placement options for unbonded concrete overlays on asphalt with widening in frost-susceptible environments



Snyder & Associates, Inc.

Materials: Tie bars should be epoxy coated with a bar size of No. 4. A 36-inch long tie bar placed at 30-inch spacing is common.

Construction: The depth of saw cuts for the longitudinal contraction joints should be T/3 for mid-slab tie bars. A saw cut depth of T/2 is recommended if tie bars are placed on the existing asphalt pavement. Tie bar inserters may be used if placement tolerances can be maintained. Otherwise, chairs may be necessary. Quality control checks should be made to confirm the location of the tie bars behind the paver. Follow details in Figure 17.6 based on UBCOA thickness.

Preventive Maintenance: Seal cracks in wet freeze/thaw climates.

Longitudinal Cracking in the Wheel Path

Cause

Longitudinal cracking in UBCOAs can result from load-related causes due to inadequate slab thickness, improper panel dimensions and joint layout, loss of bond, nonuniform support of the overlay, stripping or scouring of the asphalt due to the combination of poor drainage of the asphalt and heavy truck loading (Figure 17.7), inadequate load transfer across transverse joints (leading to high stresses at those joints under passing wheel loads), and/or panel curl/warp stresses. The most common manifestation of these combined factors is longitudinal cracking that generally develops in the wheel paths.

Figure 17.7 Erosion of asphalt due to water intrusion, causing a longitudinal crack in the wheel path of a UBCOA

Andy Bennett, Michigan DOT

Prevention

The prevention of longitudinal cracking in UBCOAs can be accomplished through the approaches described below.

Design: Provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using appropriate slab dimensions (6-by-6 feet or smaller for UBCOAs less than or equal to 6 inches thick). Take steps in design to ensure adequate support of the panel edges, including the use of dowel load transfer devices and tie bars when the overlay thickness will accommodate them. Load transfer dowels should not be used in overlays less than 7 inches thick and tie bars should not be used in overlays less than 5 inches thick. In addition, ensure that the designed joint layout does not result in panels that exceed the recommended aspect ratio of 1:1.5 (strive to keep the ratio of width to length as close to 1.0 as possible and no more than 1.5). Also, whenever possible avoid the placement of longitudinal joints within or near the wheel paths.

Materials: Consider the use of structural concrete fibers (especially for overlays that are too thin to easily accept dowel and tie bars) to hold joints tight. Structural fibers help minimize slab movement, thereby maintaining aggregate interlock in addition to increasing concrete toughness; this serves to improve long-term load transfer. Pavements prone to stripping and raveling still perform with UBCOA. Where possible, the water that may cause stripping and raveling, under heavy truck loading, should be removed through a proper drainage system. Longer lives can be expected for UBCOA where higher quality asphalt is in place. Core the existing HMA at critical locations, such as existing cracks, and visually inspect the cores for stripping.

Construction: Control the temperature (less than 120 °F) and moisture (damp with no standing water) of the existing HMA surface at the time of concrete placement. Use effective curing techniques in a timely manner for the prescribed duration. Avoid the placement of longitudinal joints directly in wheel paths. Saw cut longitudinal joints to a depth of T/3 in a timely manner.

Preventive Maintenance: Maintain sealant/filler material in the overlay joints to minimize the infiltration of water, which could induce erosion or stripping of the existing HMA layer(s).

Corner Cracking

Cause

Corner cracking (Figure 17.8) is a result of excessive corner stresses due to repeated applied loads acting in combination with one or more additional conditions or mechanisms.

Figure 17.8 Corner cracking of UBCOA

Andy Bennett, Michigan DOT

Corner cracking normally occurs at interior or exterior corners of the pavement. Interior corner cracking is generally associated with the placement of repeated wheel load directly on the slab corners, which are the weakest part of each panel (compared to loads placed at a panel edge or an interior location). Corner load placement results from placing a longitudinal joint in one of the wheel paths. Other factors that can contribute to the development of excessive corner stresses include panel movement due to stripping, shear failure of the underlying HMA resulting in a loss of support, inadequate overlay thickness (by design or as-constructed) and inadequate mechanical load transfer due to a lack of dowels, tie bars and/or aggregate interlock across the longitudinal and transverse joints that form the slab corner.

Exterior corner cracking (i.e., cracks that form in the travel lane along the lane-shoulder joint, as shown in Figure 17.8) is a result of the same mechanisms and factors listed above for interior corner cracking, except that the lane-shoulder joint is typically located near but not directly within, a wheel path. In addition, overlay panels placed adjacent to the shoulder may have very little edge support (i.e., no mechanical load transfer across the lane-shoulder joint) when the shoulder is constructed separately and/or constructed of asphalt.

Corner cracking can also occur after faulting is initiated due to increased impact loads on the downstream (depressed) slab.

Prevention

The prevention of corner cracking in UBCOA can be accomplished through the following approaches.

Design: Provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using appropriate slab dimensions (6-by-6 feet or smaller for UBCOAs less than or equal to 6 inches thick). Take steps in design to ensure adequate support of the panel edges, including the use of dowel load transfer devices and tie bars when the overlay thickness will accommodate them. In addition, ensure that the designed joint layout does not result in panels that exceed the recommended aspect ratio of 1:1.5 (strive to keep the ratio of width to length as close to 1.0 as possible and no more than 1.5) or the placement of longitudinal joints within or near the wheel paths.

Materials: Consider the use of structural concrete fibers (especially for overlays that are too thin to easily accept dowel and tie bars) to hold joints tight, thereby achieving long-term load transfer through aggregate interlock. Also, make sure that the asphalt surface to be overlaid is resistant to stripping and raveling and does not contain excessive asphalt binder, which could facilitate overlay slab movement over time.

Construction: Control the temperature (less than 120 °F) and moisture (damp with no standing water) of the existing HMA surface at the time of concrete placement. Use effective curing techniques in a timely manner for the prescribed duration. When possible, avoid the placement of longitudinal joints directly in wheel paths. Saw cut longitudinal joints to a depth of T/3 in a timely manner.

Preventive Maintenance: Maintain sealant/filler material in the overlay joints to minimize the infiltration of water, which could induce erosion or stripping of the existing HMA layer(s).

Mid-Panel Cracking

Cause

Loss of support from the underlying HMA layer(s) can lead to mid-panel cracking in the UBCOA (Figure 17.9). An inadequate thickness of existing HMA remaining after milling is the primary cause of this type of cracking. While there is no requirement for a minimum thickness of remaining HMA as in BCOA designs, there is a need for the remaining HMA to provide a stable working platform during construction and relatively uniform support for the UBCOA. In most cases, a minimum of 2 inches of HMA should remain after milling to provide a stable working platform. Mid-panel cracks may also be reflective cracks corresponding to thermal cracks in the underlying HMA.

Figure 17.9 Mid-panel cracking due to inadequate support from the underlying HMA

Tom Burnham, Minnesota DOT

Prevention

The following approaches are recommended for preventing cracking due to stripping of the existing HMA layer(s).

Design: Provide positive drainage for the pavement system, with an underdrain system. Seal joints in the UBCOA to reduce the volume of water infiltrating the underlying HMA. Set the profile grade at an elevation that ensures that at least 2 inches (preferably more) of HMA remains after milling. Provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using appropriate slab dimensions (6-by-6 feet or smaller for UBCOAs less than or equal to 6 inches thick).

Construction: When necessary, limit construction traffic on the existing HMA to prevent damage to the existing HMA. Also, limit exposure of milled asphalt surface to precipitation. Contraction joints should be sawed in a timely manner to prevent random cracking.

Transverse Cracking

Cause

Severely misaligned dowels due to movement of the dowel basket during the paving operation have been observed in some UBCOA projects (Figure 17.10). Securely anchoring dowel baskets in a nonuniform thickness of HMA can be a challenge. Fastening devices may hold securely in thicker sections of HMA and not hold in thinner sections. This can be especially critical in composite pavement sections where the existing HMA is relatively thin and the anchors extend through the HMA and penetrate the underlying portland cement concrete pavement (PCCP); the force required for the anchor to penetrate the existing PCCP can damage the surrounding HMA leaving less of the anchoring device embedded in sound material. Transverse cracking may also be reflective cracks corresponding to thermal cracks in the underlying HMA or differential movement between the UBCOA and underlying HMA during initial set

Figure 17.10 Transverse cracking due to misaligned dowels*

*The patch and the orange pavement marking (from ground penetrating radar (GPR)) clearly shows the distress occurring where the dowel bar basket moved outside the joint proper.

Matt Zeller, Concrete Paving Association of Minnesota

Prevention

Transverse cracking due to movement of the dowel basket that results in misalignment of the dowels can be mitigated in the following ways.

Design: Set the profile grade at an elevation that will provide an adequate thickness of remaining HMA for anchoring dowel baskets. Specify that shipping wires remain intact during the paving process, as this provides additional stability to the dowel basket, limiting movement and stress on the anchors.

Construction: Use an adequate number of anchors on both sides of the dowel basket to securely hold the basket in place during paving. Adjust the anchor length and driving force when nonuniform conditions are encountered. Do not cut shipping wires. Use of a dowel bar inserter eliminates the concern of misalignment due to movement of the baskets; however, proper construction techniques need to be followed to ensure that inserted dowels are placed within specified tolerances.

Transverse Joint Faulting

Cause

Conventional faulting caused by lack of load transfer, excessive fines, and the presence of water as described in Chapter 9 may occur in UBCOA pavements. However, some faulting has been observed in thin (less than or equal to 6 inches) UBCOA pavements that is not attributable to the mechanisms described in Chapter 9.

Panel movement as a result of stripping of the underlying HMA and/or a shear failure in the underlying HMA layer often precedes or is accompanied by faulting (Figure 17.11).

Figure 17.11 Faulting and panel movement of a UBCOA due to deformation of the underlying HMA

Tom Burnham, Minnesota DOT

Prevention

Regardless of the mechanism, the following general approaches in design, materials, and maintenance offer the potential for preventing the development of transverse joint faulting in UBCOAs.

Design: Provide dowel load transfer systems in overlay slabs that are thick enough to accommodate them, and confirm the structural adequacy and stability of the asphalt pavement prior to design and construction.

Materials: Consider the use of structural fiber reinforcement to enhance aggregate interlock load transfer in the overlay.

Construction: Saw all contraction joints to a depth of T/3.

Preventive Maintenance: Maintain joint filler/sealant to minimize the intrusion/collection of incompressibles and water in the overlay system. Also, periodically clean drain outlets to relieve water build-up.

Blowups

Cause

Blowups occurring in UBCOAs (Figure 17.12) can be attributed to incompressible materials in the joints or the coefficient of thermal expansion (CTE) of the concrete mixtures, as explained in Chapter 11. They may also be caused by panel movement due to stripped HMA and/or a shear failure in the HMA. Frictional forces from traffic can cause the slabs to move when these conditions are present in the underlying HMA. Aggregate interlock on sawed joints normally resists this movement but as forces increase, the longitudinal joint width widens as the wedging action translates the forces, forcing the lanes to separate to allow movement. Individual lanes become slightly skewed, resulting in a slightly longer panel length. This differential movement can lead to blowups.

Figure 17.12 Blowup of a UBCOA

Matt Zeller, Concrete Paving Association of Minnesota

Another potential cause of blowups related to panel movement is reduced friction and/or nonuniform friction between the bottom of the UBCOA and existing HMA. Depending upon the surface characteristics of the existing HMA (age, porosity, cracking, oxidation, etc.), there may be less friction between the HMA and the UBCOA as compared to a typical granular subbase.

This reduced friction can result in fewer joints deploying (cracks developing under the saw cut), due to reduced restraint stresses within the UBCOA. Although not always the case, the presence of undeployed joints can sometimes be identified in the field by a pattern of several narrow joints (nonworking) in succession separated by a wider (working joint). Coring the joints or exposing the edge of pavement is necessary to confirm whether or not they are working.

When the effective slab length is increased due to reduced subbase friction (perhaps from the designed 6 to 24 feet or longer), it means that these longer slabs will expand and contract to a greater degree than the designed smaller slabs. In addition, the reduced friction allows these long slabs to very slightly creep/slide downhill more easily than they would on a higher friction subbase.

Although designed and referred to as unbonded, all UBCOAs placed on HMA are at least partially bonded. Thus, there is some nonuniformity in the subbase friction and ease of movement. So, when these longer slabs expand, contract, and creep/slide due to the reduced subbase friction, they eventually encounter a section of the UBCOA which has a stronger bond strength than the expanding/moving section, and the stress is ultimately relieved by a blowup. Depending upon the rate of differential expansion/movement, this type of blowup (stress relief) may occur quickly or appear more gradually over time, appearing similar to severe spalling.

Other possible causes of blowups in UBCOAs include:

- Incompressibles in the joints;
- Expansion of the concrete pavement due to heavy rains, excessive heat, or a combination of both; and
- Thinner UBCOAs (less than 7 inches) can buckle easier than thicker pavements when subjected to compressive forces.

Prevention

The following approaches may be effective in preventing blowups related to movement associated with nonworking joints.

Design: Slab sizes should be appropriate for the thickness of UBCOA; the ratio of width to length should be approximately 1.0 and not exceed 1.5. Scratch milling the surface of the existing HMA may result in more uniform subbase friction. Specify the depth of saw cut to be one-third of the design thickness (T/3). Specify sealing joints to prevent incompressible materials from filing the joints.

Construction: Cut all joints to T/3. Minimize cold weather paving for UBCOA pavements less than 7 inches thick. Seal all joints. Saw cut expansion joints at the specified intervals. Cut transverse joints full depth every 12 feet on UBCOA pavements with minimal truck traffic.

Treatment: When excessive slab movement or blowups occur, an unproven strategy that may mitigate future blowups is to saw full depth across the full width of the pavement at approximately 300-foot intervals.

A summary of the most common contributing causes and prevention or mitigation measures for UBCOA-specific distresses is provided in Table 17.3.

Table 17.3 Summary of the most common contributing causes and prevention/mitigation measures for UBCOA-specific distresses

UBCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Longitudinal Cracking in Tied Shoulders and Widened Sections	Differential movement (heaving) of materials underlying shoulders and/or widened sections Tie bar placed at the bottom of the UBCOA slab Tie bar size larger than No. 4 Too many tie bars per slab	Tie bar placed at neutral axis, if possible Utilize structural fibers in lieu of tie bars	Maximum tie bar size of No. 4	Chair or insert bars to specified tolerances QC checks to confirm tie bar location behind the paver Saw all contraction joints to a depth of T/3	Seal cracks in wet-freeze climates
Longitudinal Cracking in Wheel Path	Inadequate slab thickness Improper slab dimensions Inadequate load transfer Excessive curling/warping Asphalt stripping and/or scouring Inadequate tie bar placement, particularly with tied shoulders Inadequate subbase stabilization under widened sections	Adequate overlay thickness Appropriate slab dimensions for the design thickness Load transfer devices when thickness is greater than or equal to 7 in. Avoid placing longitudinal joints in the wheel path Provide adequate drainage of the existing asphalt pavement Tie bar placed at neutral axis if possible Provide stable and non-expansive subbase materials under widened sections	Structural fibers may increase load transfer in thin overlays Existing HMA should be resistant to stripping	Control the temperature and moisture of the HMA surface at time of paving Adequate curing Avoid placing longitudinal joints in the wheel path Saw cut longitudinal joints to T/3	Maintain joint sealant/filler

UBCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Corner Cracking	Longitudinal joint in wheel path Inadequate slab thickness Inadequate load transfer Lack of edge support Panel movement	Avoid placing longitudinal joints in the wheel path Adequate overlay thickness Load transfer devices when thickness is greater than or equal to 7 in.	Structural fibers may increase load transfer in thin overlays Existing HMA should be resistant to stripping Existing HMA should not contain excessive binder	Place shoulder integral with mainline to improve edge support Control the temperature and moisture of the HMA surface at time of paving Adequate curing Avoid placing longitudinal joints in the wheel path Saw cut longitudinal joints to T/3	Maintain joint sealant/filler
Mid-Panel Cracking	Inadequate thickness of existing HMA after milling Loss of support due to poor drainage and subsequent stripping of HMA Reflective cracking of underlying thermal cracks in the HMA	Set profile grade to ensure adequate thickness of existing HMA remains after milling	N/A	When necessary, limit construction traffic on the existing HMA to prevent damage to the existing HMA Saw cut transverse joints to T/3	N/A

UBCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Transverse Cracking	Misaligned dowels due to movement of the dowel basket Differential movement between the UBCOA and underlying HMA during initial set Reflective cracking of underlying thermal cracks in the HMA	Set profile grade to ensure adequate thickness of existing HMA remains after milling Specify that shipping wires should not be cut	N/A	Securely anchor baskets Adjust anchoring for nonuniform conditions Do not cut shipping wires on the baskets Utilize a dowel-bar-inserter Saw cut transverse joints to T/3	N/A
Transverse Joint Faulting	Deformation of the underlying asphalt under repeated loading Slab movement Erosion of the asphalt binder and fines underneath the joint Panel movement	Load transfer devices when thickness is greater than 7 in. Confirm stability of the HMA prior to final design	Structural fibers may increase load transfer in thin overlays	N/A	Maintain joint sealant/filler

UBCOA Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Blowups	Undeployed joints in the UBCOA Nonuniform subbase friction Panel movement Incompressibles in the joints Expansion due to heavy rain, excessive heat, or both Thin overlays	Appropriate slab dimensions for the design thickness Scratch milling of the surface to provide more uniform subbase friction Specify sealing joints to prevent incompressibles from filing the joints Provide expansion joints at appropriate intervals A full-depth transverse saw cut every 12 ft on roadways with minimal truck traffic may reduce the potential for blowups Ensure that the underlying HMA is well drained	N/A	Saw cut joints to T/3 Minimize cold weather paving for UBCOA pavements less than 7 in. thick Seal all joints	N/A

6. Treatment and Repairs

Maintenance and repair of the distresses discussed in this chapter consist of common procedures. Best practices for these methods can be found in the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

The applicability of these treatments is summarized in Table 17.4.

Table 17.4 Treatment options for UBCOA-specific distresses

Distress	Treatment: Underdrain Maintenance	Treatment: Resealing Joints	Treatment: Diamond Grinding	Treatment: Rout and Seal Cracks/Joints	Treatment: Full-Depth Patching	Do Nothing
Longitudinal Cracking in Tied Shoulders and Widened Sections	✓			✓		✓
Longitudinal Cracking in Wheel Path		✓		✓	✓	✓
Corner Cracking		✓		✓	✓	✓
Mid-Panel Cracking	✓	✓		✓	✓	✓
Transverse Cracking				✓	✓	✓
Transverse Joint Faulting	✓	✓	✓			✓
Blowups	✓	✓			✓	

7. References

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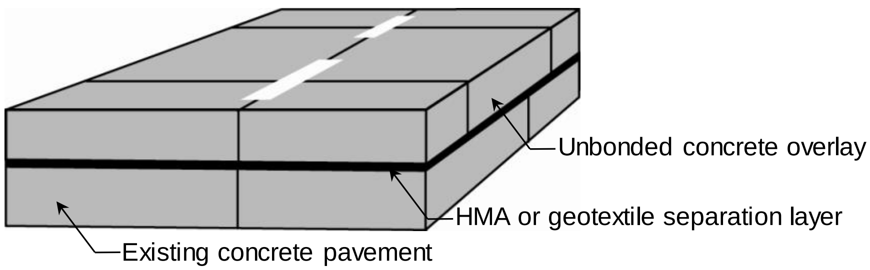
Smith, K., D. Harrington, L. Pierce, P. Ram, and K. Smith. 2014. *Concrete Pavement Preservation Guide*. Second Edition. National Concrete Pavement Technology Center, Ames, IA. http://www.cptechcenter.org/technical-library/documents/preservation_guide_2nd_ed_508_final.pdf.

CHAPTER 18. UNBONDED CONCRETE OVERLAY ON CONCRETE (UBCOC)

1. Description

Unbonded concrete overlay on concrete (UBCOC) can be found on all functional classifications of roadways. By definition, UBCOCs consist of a new portland cement concrete surface placed over an existing concrete pavement. The two concrete layers are separated by an asphalt or geotextile interlayer that is designed to provide isolation, bedding, and drainage (Figure 18.1).

Figure 18.1 Illustration of an unbonded concrete overlay on concrete (UBCOC)



Harrington and Fick 2014, National Concrete Pavement Technology Center

The design of UBCOCs treat the existing pavement and separation layer as a stiff base, and not as a part of the pavement thickness. Thus, UBCOC thicknesses have traditionally been slightly thinner than what would be required by traffic estimates for a full-depth concrete pavement placed on a granular subbase. Typical thicknesses are greater than 6 inches, although some UBCOCs less than 6 inches thick have been constructed on lower traffic volume roadways. These concrete pavements can be designed as jointed plain concrete pavement (JPCP), (with or without load transfer) or continuously reinforced concrete pavement (CRCP). Joint spacing for JPCP unbonded overlays should be a function of the design thickness (Harrington and Fick 2014).

Summary of UBCOC Distresses

Each of the distresses covered in Chapters 2 through 13 should be consulted for additional information on the identification, causes, evaluation, and treatment of the observed distresses found in UBCOCs. However, there are some of these distresses which may manifest themselves differently in UBCOCs. So, for a given distress observed in a UBCOC the cause(s) may be as described in the appropriate distress chapter for a non UBCOC pavement; or the cause may be related to the design and construction of the UBCOC. This chapter deals exclusively with unique causes of distresses observed in UBCOCs.

Example photos of these UBCOC distresses are shown in Figure 18.2. These distresses and their associated causes are the focus of this chapter.

Figure 18.2 Common UBCOC-specific distresses

a. Longitudinal cracking in the wheel path of UBCOC with tied shoulders and widened sections

Gary Fick, Trinity Management Services, Inc.

b. Longitudinal cracking in wheel path in UBCOC

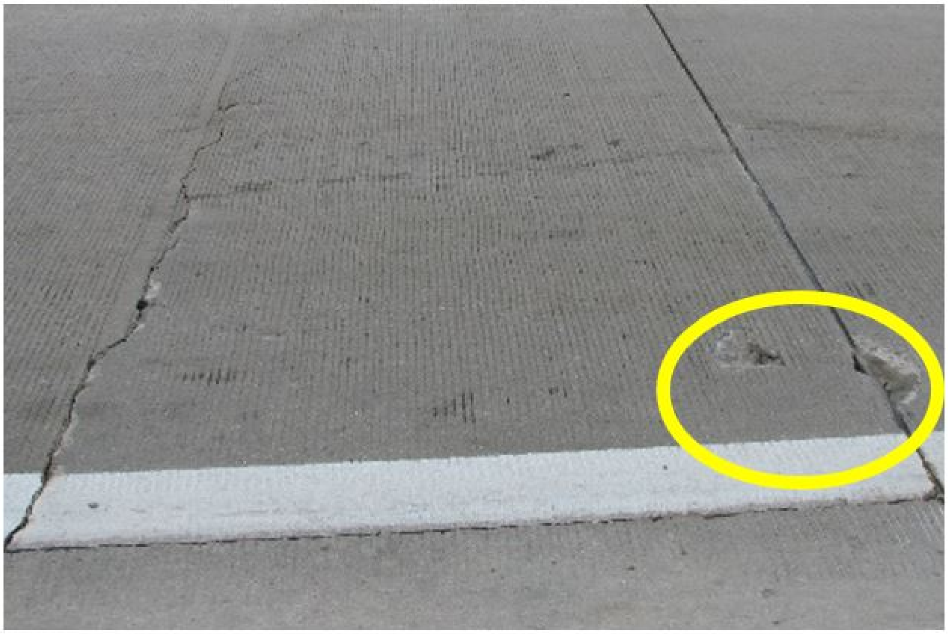
Tom Burnham, Minnesota DOT

c. Corner cracking of UBCOC
Gary Fick, Trinity Management Services, Inc.

d. Transverse reflective cracking of UBCOC
Tom Burnham, Minnesota DOT

e. Irregular reflective cracking of UBCOC
Andy Bennett, Michigan DOT

f. Mid-panel cracking of UBCOC
Matt Zeller, Concrete Paving Association of Minnesota



g. Transverse cracking of UBCOC due to severely misaligned dowels, yellow oval shows the exposed end of a misaligned dowel
Andy Bennett, Michigan DOT

h. Transverse joint faulting of UBCOC
Tom Burnham, Minnesota DOT

i. Blowups
Tom Burnham, Minnesota DOT

2. Severity

Table 18.1 summarizes the severity and measurement of distresses which are unique to UBCOCs. The information in the table is derived from the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2014).

Table 18.1 Proposed severity levels and measurement for UBCOC-specific distresses

Distress	Description	Severity Levels	Extent Levels
Longitudinal Cracking in the Wheel Path of a UBCOC with Tied Shoulders and Widened Sections **Longitudinal Cracking in Wheel Path** **Mid-Panel Cracking** **Transverse Cracking**	A longitudinal crack is a full depth fracture of the concrete overlay, orientated roughly parallel to the pavement centerline or the lane - shoulder joint. A transverse crack is a full-depth fracture of the concrete overlay that develops roughly perpendicular to the centerline.	**Low**: Crack is less than 1/16 in. (1.5 mm) wide, spalling over less than 10% of length, adjacent slab fragments are not broken into multiple pieces, and there is no faulting **Medium:** Crack width is 1/16 to 1/8 in. (1.5 to 3.0 mm) wide, or crack is spalled (low severity) over more than 10% of length, adjacent slab fragments are not broken into multiple pieces, and faulting is less than 1/2 in. (13 mm) **High**: Crack width exceeds 1/8 in. (3mm), or exhibits medium- or high-severity spalling over greater than 10% of length, or either adjacent slab fragment is broken into two or more pieces and may be loose, or faulting greater than or equal to 1/2 in. (13 mm)	Record the number of panels with transverse and longitudinal cracking and record the number of transverse and longitudinal cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity crack and as a patch.
Corner Cracking	A corner crack is a full-depth fracture of the concrete overlay that intersects adjacent transverse and longitudinal joints at an angle of approximate 45° with the direction of the traffic. The lengths of the sides are always less than one-half the width of the slab on each side of the corner.	**Low**: Crack is less than 1/16-in. (1.5 mm) wide, spalling over less than 10% of crack length, corner fragment is not broken into multiple pieces, and there is no measurable faulting **Medium**: Crack is spalled (low severity) over more than 10% of length, corner fragment is not broken into multiple pieces, and faulting is less than 1/2 in. (13 mm) **High**: Crack is spalled (medium or high severity) over more than 10% of length, corner fragment is broken into two or more pieces and may be loose, or faulting is greater than or equal to 1/2 in. (13 mm)	Record the number of panels with corner cracks and record the number of corner cracks at each severity level. Areas that have been replaced with repair material (rigid or flexible) should be rated as both a high-severity corner crack and as a patch.

Distress	Description	Severity Levels	Extent Levels
Transverse Joint Faulting	A difference of elevation across a joint or a crack.	**Low**: Less than or equal to 1/8 in. (3mm) fault **Medium**: 1/8 to 3/8 in. (3 to 9.5 mm) fault **High**: Greater than 3/8 in. (9.5 mm) fault	**Occasional**: Faulting occurs along less than 20% of the joints and cracks **Frequent**: Faulting occurs along 20 to 50% of the joints and cracks **Extensive**: Greater than 50% of the joints and cracks are faulted
Blowups	The result of localized upward movement of the concrete overlay or shattering of the overlay along the transverse joint or crack.	N/A	N/A

Source: Miller and Bellinger 2014

3. Testing

The mechanisms involved in the most common distresses that are unique to UBCOCs are listed below.

- Inadequate evaluation of existing conditions, resulting in inadequate design
- Load-related cracking due to inadequate as-built overlay thickness and/or actual traffic being greater than that assumed in design
- Placement of longitudinal joints in the wheel paths
- Improper tie bar size and placement in shoulder and widened sections
- Loss of support due to inadequate drainage resulting in stripping of the hot mix asphalt (HMA) interlayer
- Nonworking transverse joints
- Misplaced load transfer dowels
- Panel movement due to stripping and/or shear failure of the underlying HMA

These mechanisms may be identified through field surveys and coring where appropriate, and as described below.

Distress/Condition Survey

Distress and condition surveys often reveal the probable mechanism of the distress by its form and location (e.g., transverse panel crack aligned with transverse crack in adjacent asphalt shoulder). Field site visits are also an opportunity to verify wheel paths relative to longitudinal joint locations.

Coring

Coring is useful in determining actual pavement layer thicknesses in areas of distress for structural evaluation and assessment.

Examination

Direct visual examination of the concrete pavement surface and cracking pattern can play a large role in determining the type and causes of distress in existing pavement.

Nondestructive Testing

There are a number of nondestructive methods that may be helpful in locating load transfer dowels and tie bars. Depending upon the distress observed (e.g., cracking near the joint, longitudinal cracking, transverse joint faulting, etc.), locating the embedded steel items may assist in diagnosing the root cause of the distress. Common nondestructive methods available include those listed below.

- Magnetic imaging tomography scanning for dowel bars; note this method may not be suitable for dowels placed in baskets if the shipping wires were left intact, or if the existing HMA contains slag aggregate
- MIT scan T2 for tie bars
- Ground penetrating radar to find the general location of embedded steel
- Pachometer to find the general location of embedded steel

The applicability of these devices to specific distresses is covered in previous chapters. Manufacturer's recommendations should be consulted for proper operation of these devices.

More detailed description of specific testing procedures can be found in Chapter 19.

4. Identification of Causes

The causes of most UBCOC-specific distresses can generally be traced to improper evaluation of the existing conditions, certain design and construction issues, and (often to a lesser extent) some material issues. Several of these factors are listed and described briefly in Table 18.2. More detailed discussions of appropriate distress causes unique to UBCOCs follow Table 18.2.

Table 18.2 Summary of causes of UBCOC-specific distresses

Distress	Category	Description of Causes Unique to UBCOC
Longitudinal Cracking in Tied Shoulders and Widened Sections	Physical	Differential movement (heaving) of materials underlying shoulders and/or widened sections Tie bar placed at the bottom of the UBCOC slab Too many tie bars per slab Inadequate compaction in widened section
Longitudinal Cracking in Tied Shoulders and Widened Sections	Material/Chemical	Tie bar size larger than No. 4
Longitudinal Cracking in Wheel Path	Physical	Inadequate slab thickness Improper slab dimensions or wheel overloading Inadequate base support or Inadequate load transfer Asphalt separation layer stripping and/or scouring Inadequate tie bar placement, particularly with tied shoulders Excessive curling/warping
Corner Cracking	Physical	Longitudinal joint near wheel path Inadequate slab thickness Inadequate load transfer Lack of edge support Panel movement
Transverse Reflective Cracking	Physical	Inadequate separation layer Failure to match existing expansion joint locations
Irregular Reflective Cracking	Physical	Wrinkled geotextile separation layer

Distress	Category	Description of Causes Unique to UBCOC
Mid-Panel Cracking	Physical	Loss of support due to poor drainage and subsequent stripping of the HMA separation layer Inadequate overlay thickness and/or wheel overloading
Mid-Panel Cracking	Material/Chemical	HMA separation layer susceptible to stripping
Transverse Cracking	Physical	Misaligned dowels due to movement of the dowel basket Inappropriate slab dimensions Structural inadequacy resulting in premature fatigue Differential movement between the UBCOC and underlying pavement
Transverse Joint Faulting	Physical	Deformation of the HMA separation layer under repeated loading Slab movement Erosion of the asphalt binder and fines underneath the joint Inadequate load transfer Panel movement
Blowups	Physical	Undeployed joints in the UBCOC Panel movement

5. Evaluation

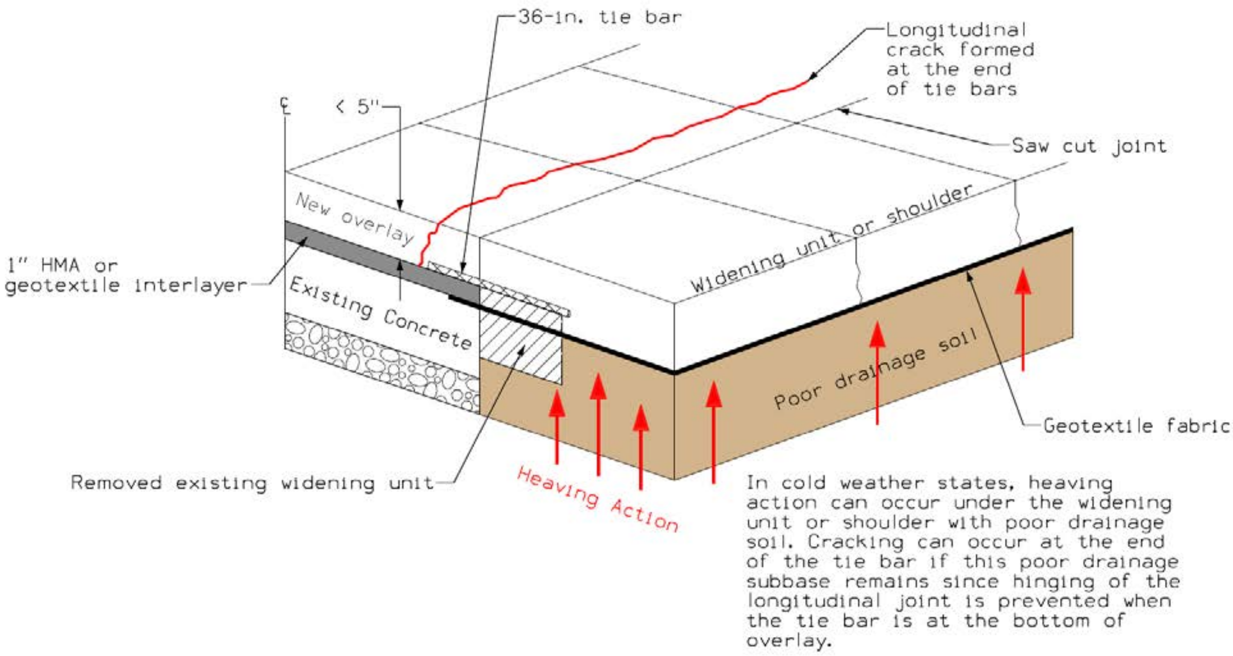
The following sections describe the specific causes of each of the previously listed UBCOC distresses and briefly discusses approaches for their prevention.

Longitudinal Cracking in Tied Shoulders and Widened Sections

Cause

Some UBCOCs have been placed over older narrow sections of roadway that have either been previously widened or are widened during the construction of the UBCOC. Similar to these widened sections, other UBCOCs have been constructed with tied concrete shoulders where the underlying shoulder materials offer less support to the UBCOC than the existing mainline pavement section. In both cases, when movement occurs in the materials underlying the shoulder/widened section, or an excessive load is applied, the outside longitudinal joint between the mainline and widening unit or shoulder is not allowed to hinge properly, a longitudinal crack can form at the end of the tie bars. Many of these UBCOC widening units were designed and constructed with a tie bar stapled to the top of the HMA separation layer (Figure 18.3) when the overlay is less than 5 inches thick.

Figure 18.3 Detail showing how inadequate subbase stability and excessive loads under widened sections contributes to longitudinal cracking at the end of tie bars in cold weather regions



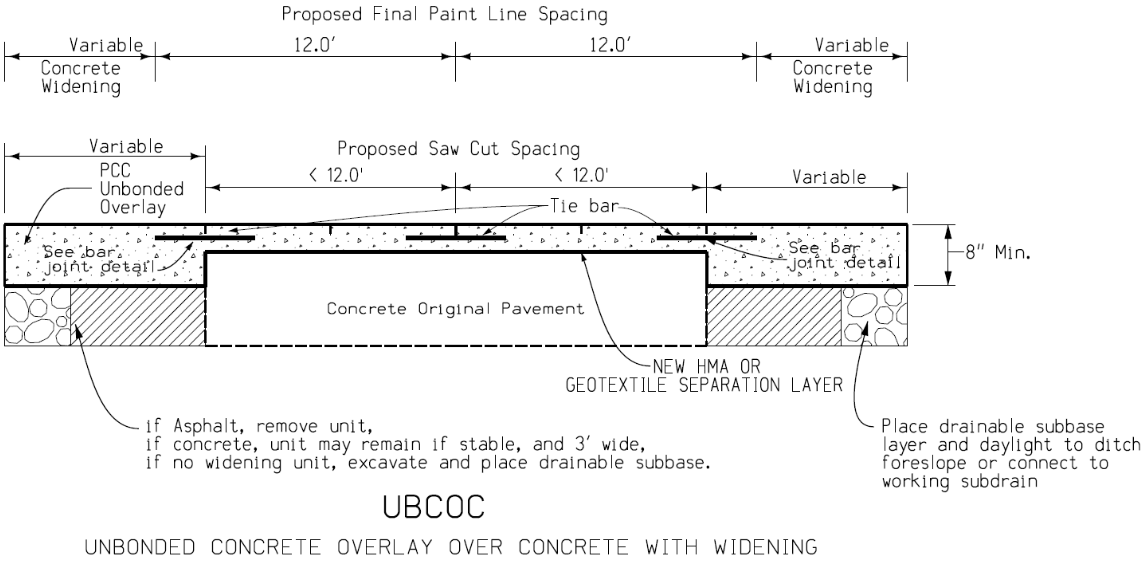
Snyder & Associates, Inc.

Prevention

Interim guidance recommends placing the tie bar at the neutral axis (mid-depth) of the UBCOC whenever possible, or alternatively using structural fibers to provide post-cracking integrity of the UBCOC.

Design: Provide adequate overlay thickness (greater than 4.5 inches) to allow placement of bars with a minimum of 2-inch cover above and below the tie bars to allow the longitudinal joint to hinge or utilize structural fiber reinforcement (see Figure 18.4). Also, provide adequate subbase support (stable, drainable, and non-expansive) under the widened or shoulder section, particularly if the overlay thickness is less than 4.5 inches and requires tie bars to be fastened to the asphalt pavement. In this situation, it is important that no heaving of the subgrade or cracking occur. When the widening unit is not as thick as the combined new overlay and existing pavement thickness, the subbase stability must be taken into consideration. Under heavy truck traffic loading, the granular subbase may need to be stabilized (e.g., cement-treated base).

Figure 18.4 Cross-section of a proposed UBCOC design with paved widening/shoulder unit and placement of separation layer and tie bars



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Materials: Maximum bar size should be No. 4.

Construction: The depth of saw cuts for the longitudinal contraction joints should be T/3; tie bar inserters may be used if placement tolerances can be maintained. Otherwise, chairs may be necessary. Quality control checks should be made to confirm location of the tie bars behind the paver.

Preventive Maintenance: Seal cracks in wet freeze/thaw climates.

Longitudinal Cracking in the Wheel Path

Cause

Longitudinal cracking in UBCOCs can result from load-related causes due to inadequate slab thickness, improper panel dimensions and joint layout, nonuniform support of the overlay, stripping or scouring of the HMA separation layer due to the combination of poor drainage of the HMA and heavy truck loading (Figure 18.5), inadequate load transfer across transverse joints (leading to high stresses at those joints under passing wheel loads), and/or panel curl/warp stresses. The most common manifestation of these combined factors is longitudinal cracking that generally develops in the wheel paths.

Figure 18.5 Erosion of asphalt due to water intrusion, stripping, and erosion of the HMA, causing a longitudinal crack in the wheel path of a UBCOC

Andy Bennett, Michigan DOT

Prevention

The prevention of longitudinal cracking in UBCOCs can be accomplished through the approaches described below.

Design: Provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using appropriate slab dimensions (6-by-6 feet or smaller for UBCOCs less than or equal to 6 inches thick). Take steps in design to ensure adequate support of the panel edges, including the use of dowel load transfer devices and tie bars when the overlay thickness will accommodate them. Load transfer dowels should not be used in overlays less than 7 inches thick and tie bars should not be used in overlays less than 5 inches thick. In addition, ensure that the designed joint layout does not result in panels that exceed the recommended aspect ratio of 1:1.5 (strive to keep the ratio of width to length as close to 1.0 as possible and no more than 1.5). Also, whenever possible avoid the placement of longitudinal joints within or near the wheel paths.

Materials: Consider the use of structural concrete fibers (especially for overlays that are too thin to easily accept dowel and tie bars) to hold joints tight. Structural fibers help minimize slab movement, thereby maintaining aggregate interlock in addition to increasing concrete toughness; this serves to improve long-term load transfer. Where possible, the water that may cause potential stripping and raveling in the HMA separation layer under heavy truck loading, should be removed through a proper drainage system. Longer lives can be expected for UBCOCs where a HMA separation layer that is resistant to stripping is used.

Construction: Control the surface temperature (less than 120 °F) and moisture (damp with no standing water) of the existing HMA separation layer at the time of concrete placement. Use effective curing techniques in a timely manner for the prescribed duration. Avoid the placement of longitudinal joints directly in wheel paths. Saw cut longitudinal joints to a depth of T/3 in a timely manner.

Preventive Maintenance: Maintain sealant/filler material in the overlay joints to minimize the infiltration of water, which could induce erosion or stripping of the HMA separation layer.

Corner Cracking

Cause

Corner cracking (Figure 18.6) is a result of excessive corner stresses due to repeated applied loads acting in combination with one or more additional conditions or mechanisms.

Figure 18.6 Interior corner cracking (left) and exterior corner cracking (right)

Gary Fick, Trinity Management Services, Inc. (left), Matt Zeller, Concrete Paving Association of Minnesota (right)

Interior corner cracking is generally associated with the placement of repeated wheel loads directly on the slab corners, which are the weakest part of each panel (compared to loads placed at a panel edge or an interior location). Corner load placement results from placing a longitudinal joint in one of the wheel paths. Other factors that can contribute to the development of excessive corner stresses include the following:

- Panel movement due to stripping
- Shear failure of the underlying HMA resulting in a loss of support
- Inadequate overlay thickness (by design or as-constructed)
- Inadequate mechanical load transfer due to a lack of dowels, tie bars, and/or aggregate interlock across the longitudinal and transverse joints that form the slab corner.

Exterior corner cracking (i.e., cracks that form in the travel lane along the lane-shoulder joint) is a result of the same mechanisms and factors listed above for interior corner cracking, except that the lane-shoulder joint is typically located near but not directly within, a wheel path. In addition, overlay panels placed adjacent to the shoulder may have very little edge support (i.e., no mechanical load transfer across the lane-shoulder joint) when the shoulder is constructed separately and/or constructed of asphalt.

Corner cracking can also occur after faulting is initiated due to increased impact loads on the downstream (depressed) slab.

Prevention

The prevention of corner cracking in UBCOC can be accomplished through the approaches described below.

Design: Provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using appropriate slab dimensions (6-by-6 feet or smaller for UBCOCs less than or equal to 6 inches thick). Take steps in design to ensure adequate support of the panel edges, including the use of dowel load transfer devices and tie bars when the overlay thickness will accommodate them. In addition, ensure that the designed joint layout does not result in panels that exceed the recommended aspect ratio of 1:1.5 (strive to keep the ratio of width to length as close to 1.0 as possible and no more than 1.5) or the placement of longitudinal joints within or near the wheel paths.

Materials: Consider the use of structural concrete fibers (especially for overlays that are too thin to easily accept dowel and tie bars) to hold joints tight, thereby achieving long-term load transfer through aggregate interlock. Also, make sure that the asphalt surface to be overlaid is resistant to stripping and raveling and does not contain excessive asphalt binder, which could facilitate overlay slab movement over time.

Construction: Control the surface temperature (less than 120 °F) and moisture (damp with no standing water) of the HMA separation layer at the time of concrete placement. Use effective curing techniques in a timely manner for the prescribed duration. When possible, avoid the placement of longitudinal joints directly in wheel paths. Saw cut longitudinal joints to a depth of T/3 in a timely manner.

Preventive Maintenance: Maintain sealant/filler material in the overlay joints to minimize the infiltration of water, which could induce erosion or stripping of the HMA separation layer.

Transverse Reflective Cracking

Cause

Movement from existing joints and/or cracks in the underlying concrete pavement can reflect through to the UBCOC when the HMA separation layer is not thick enough to fully isolate the UBCOC from the existing concrete (Figure 18.7). Existing expansion joints in the underlying concrete typically allow for more movement than contraction joints, and thus can be reflected through the HMA separation layer into the UBCOC.

Figure 18.7 Transverse reflective cracking of UBCOC

Andy Bennett, Michigan DOT

Prevention

Design: Provide an adequate thickness of HMA separation layer (1 to 1.5 inches). Place expansion joints in the UBCOC directly above expansion joints in the existing concrete pavement.

Construction: Match expansion joint locations in the underlying concrete pavement.

Irregular Reflective Cracking

Cause

It has been noted that wrinkles in a geotextile separation layer have manifested in irregular cracking in UBCOC (Figure 18.8).

Figure 18.8 Irregular reflective cracking due to wrinkled geotextile separation layer

Andy Bennett, Michigan DOT

Prevention

Construction: Secure the geotextile fabric with adequate anchors and/or adhesive to prevent wrinkling, especially in areas where vehicles are turning on the geotextile separation layer.

Stripping of HMA Separation Layer Causing Mid-Panel Cracking

Cause

Loss of support from the underlying HMA separation layer can lead to mid-panel cracking in the UBCOC. Stripping of the HMA separation layer is the primary cause of this type of distress. On most projects, a nominal 1-inch HMA separation layer provides adequate coverage over irregularities in existing concrete pavement. However, asphalt stripping of the HMA separation layer can occur under high truck traffic volumes through repetitive loading, particularly in the presence of trapped water in the separation layer.

Usually, the stripping takes several years to develop. Figure 18.9 is an example of mid-panel cracking due to stripping.

Figure 18.9 Mid-panel cracking due to inadequate support (stripping) of the HMA separation layer

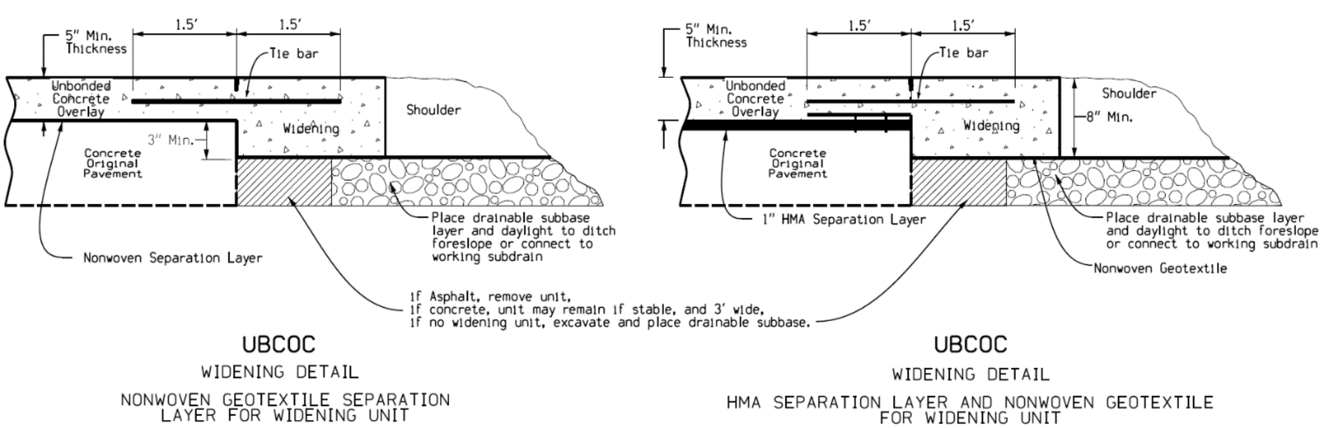
Dan DeGraaf, Michigan Concrete Association

Prevention

The following approaches are recommended for preventing cracking due to stripping of the HMA separation layer. (See Figure 18.10.)

Design: Provide positive drainage for the pavement system, with an underdrain system or by daylighting the HMA separation layer through the use of nonwoven geotextile outlet shown on the right-hand side of the figure below. Seal joints in the UBCOC to reduce the volume of water infiltrating the HMA separation layer. Provide adequate thickness of the overlay and reduce potential curl/warp stresses to negligible levels by using appropriate slab dimensions (6-by-6 feet or smaller for UBCOCs less than or equal to 6 inches thick).

Figure 18.10 Detail showing drainage outlets for geotextile or HMA separation layer for UBCOC



Snyder & Associates, Inc.

Materials: Utilize a HMA separation layer that is more resistant to stripping. In an effort to reduce the scour pore pressure and increase stability, some states modify the asphalt mixture that make it somewhat more porous than the standard asphalt mix. In addition, you may incorporate anti-strip additives such as lime in the asphalt. Lime has been found to be more effective than liquid anti-stripping additives. An alternate to the HMA separation layer is the use of a nonwoven geotextile separation layer.

Construction: Avoid crushing the underdrain system during construction.

Preventive Maintenance: Periodically clean underdrain systems.

Transverse Cracking Due to Severely Misaligned Load Transfer Dowels

Causes

Severely misaligned dowels due to movement of the dowel basket during the paving operation have been observed in some UBCOC projects (Figure 18.11). Securely anchoring dowel baskets in a nonuniform thickness of HMA separation layer can be a challenge. Fastening devices may hold securely in thicker sections of HMA and not hold in thinner sections. This can be especially critical where the HMA separation layer is used to make cross-slope adjustments resulting in variable thickness of HMA where the anchors extend through the HMA and penetrate the underlying portland cement concrete pavement (PCCP); the force required for the anchor to penetrate the existing PCCP can damage the surrounding HMA leaving less of the anchoring device embedded in sound material.

Figure 18.11 Transverse cracking due to misaligned dowels

Andy Bennett, Michigan DOT

Prevention

Transverse cracking due to movement of the dowel basket which results in misalignment of the dowels can be mitigated in the following ways.

Design: Provide a nominal thickness of HMA separation layer and make cross-slope adjustments in the UBCOC. Specify that shipping wires remain intact during the paving process as this provides additional stability to the dowel basket, limiting movement and stress on the anchors.

Construction: Use an adequate number of anchors on both sides of the dowel basket to securely hold the basket in place during paving. Adjust the anchor length and driving force when nonuniform conditions are encountered. Do not cut shipping wires. Use of a dowel bar inserter eliminates the concern of misalignment due to movement of the baskets; however, proper construction techniques need to be followed to ensure that inserted dowels are placed within specified tolerances.

Transverse Joint Faulting

Cause

Conventional faulting caused by lack of load transfer, excessive fines and the presence of water, as described in Chapter 9, may occur in UBCOC pavements. However, some faulting has been observed in thin (less than 6 inches) UBCOC pavements that is not attributable to the mechanisms described in Chapter 9.

Panel movement as a result of stripping of the HMA separation layer and/or a shear failure in the underlying HMA layer often precedes or is accompanied by faulting (Figure 18.12).

Figure 18.12 Faulting and panel movement of a concrete overlay due to deformation of the underlying HMA

Brad Letcher, South Dakota DOT

Prevention

Regardless of the mechanism, the following general approaches in design, materials, and maintenance seem to offer potential for preventing the development of transverse joint faulting in UBCOCs.

Design: Provide dowel load transfer systems in overlay slabs that are thick enough to accommodate them, and confirm the structural adequacy and stability of the asphalt pavement prior to design and construction.

Materials: Consider the use of structural fiber reinforcement to enhance aggregate interlock load transfer in the overlay.

Preventive Maintenance: Maintain joint filler/sealant to minimize the intrusion/collection of incompressible materials and water in the overlay system. Also, periodically clean drain outlets to relieve water build-up.

Blowups

Cause

Blowups occurring in UBCOCs (Figure 18.13) can be attributable to incompressibles in the joints or the coefficient of thermal expansion (CTE) of the concrete mixtures as explained in Chapter 11. They may also be caused by panel movement due to stripped HMA and/or a shear failure in the HMA. Frictional forces from traffic can cause the slabs to move when these conditions are present in the underlying HMA. Aggregate interlock on sawed joints normally resists this movement but as forces increase, the longitudinal joint width will widen as the wedging action translates the forces, forcing the lanes to separate to allow movement. Individual lanes become slightly skewed, resulting in a longer panel length. This differential movement can lead to blowups.

Figure 18.13 Blowup of an UBCOC

John Brunkhorst, McLeod County, Minnesota

Another potential cause of blowups related to panel movement is reduced friction and/or nonuniform friction between the bottom of the UBCOC and HMA separation layer. Depending upon the surface characteristics of the HMA separation layer, there may be less friction between the HMA and the UBCOC as compared to a typical granular subbase.

This reduced friction can result in fewer joints deploying (cracks developing under the saw cut), due to reduced restraint stresses within the UBCOC. Although not always the case, the presence of undeployed joints can sometimes be identified in the field by a pattern of several narrow joints (nonworking) in succession separated by a wider (working joint). Coring the joints or exposing the edge of pavement is necessary to confirm whether they are working or not.

When the effective slab length is increased due to reduced subbase friction (perhaps from the designed 6 to 24 feet or longer), it means that these longer slabs will expand and contract to a greater degree than the designed smaller slabs. In addition, the reduced friction also allows these long slabs to very slightly creep/slide downhill more easily than they would on a higher friction subbase.

Although designed and referred to as unbonded, all UBCOCs placed on HMA separation layer are at least partially bonded. Thus, there is some nonuniformity in the subbase friction and ease of movement. So, when these longer slabs expand, contract, and creep/slide due to the reduced subbase friction, they eventually encounter a section of the UBCOC, which has a stronger bond strength than the expanding/moving section, and the stress is ultimately relieved by a blowup. Depending upon the rate of differential expansion/movement, this type of blowup (stress relief) may occur quickly or may appear more gradually over time, appearing similar to severe spalling.

Other possible causes of blowups in UBCOCs include:

- Incompressibles in the joints;
- Expansion of the concrete pavement due to heavy rains, excessive heat, or a combination of both; and
- Thinner UBCOCs (less than 7 inches) can buckle easier than thicker pavements when subjected to compressive forces.

Prevention

The following approaches may be effective in preventing blowups related to movement associated with nonworking joints.

Design: Slab sizes should be appropriate for the thickness of UBCOC; the ratio of width to length should be approximately 1.0 and not exceed 1.5. Specify the depth of saw cut to be one-third of the design thickness (T/3). Specify sealing joints to prevent incompressible materials from filling the joints.

Construction: Cut all joints to T/3. Minimize cold weather paving for UBCOC pavements less than 7 inches thick. Seal all joints. Saw cut expansion joints at the specified intervals. Cut transverse joints full depth every 12 feet on UBCOC pavements with minimal truck traffic.

Treatment: When excessive slab movement or blowups occur, an unproven strategy that may mitigate future blowups is to saw full depth across the full width of the pavement at approximately 300-foot intervals. This strategy should be first tried on a limited basis until proven to work.

Other Distresses Observed on UBCOCs Less Than or Equal to 6 Inches Placed on Geotextile

Although this is not a pavement distress or a structural performance issue, there has been at least one thin UBCOC constructed on a geotextile separation layer that exhibited a thumping noise after opening to traffic. The noise was generated by the slabs rocking against each other at the transverse joints under traffic loading. This was attributed to vertical movement of the small slabs due to an uncompressed geotextile separation layer. The thumping eventually subsided after a few months. The latest guidance for specifying geotextiles as a separation layer recommends using a lighter weight (thinner) geotextile for UBCOCs less than or equal to 6 inches thick to mitigate this issue.

Table 18.3 summarizes the most common contributing causes and prevention/mitigation strategies for UBCOC-specific distresses.

Table 18.3 Summary of the most common contributing causes and prevention/mitigation measures for UBCOC-specific distresses

UBCOC Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Longitudinal Cracking in Tied Shoulders and Widened Sections	Differential movement (heaving) of materials underlying shoulders and/or widened sections Tie bar placed at the bottom of the UBCOC slab Tie bar size larger than No. 4 Too many tie bars per slab Inadequate compaction in widened section	Tie bar placed at neutral axis if possible Utilize structural fibers in lieu of tie bars Adequate compaction of stable (non-expansive) subbase	Maximum tie bar size of No. 4	Chair or insert bars to specified tolerances Quality control checks to confirm tie bar location behind the paver Saw all contraction joints to a depth of T/3	Seal cracks in wet-freeze climates
Longitudinal Cracking in Wheel Path	Inadequate slab thickness Improper slab dimensions Inadequate load transfer Excessive curling/warping Asphalt stripping and/or scouring Inadequate tie bar placement, particularly with tied shoulders Excessive curling/warping	Adequate overlay thickness Appropriate slab dimensions for the design thickness Load transfer devices when thickness is greater than or equal to 7 in. Avoid placing longitudinal joints in the wheel path Provide adequate drainage of the HMA separation layer Tie bar placed at neutral axis if possible	Structural fibers may increase load transfer in thin overlays HMA separation layer should be resistant to stripping	Control the temperature and moisture of the HMA surface at time of paving Adequate curing Avoid placing longitudinal joints in the wheel path Saw cut longitudinal joints to T/3	Maintain joint sealant/filler

UBCOC Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Corner Cracking	Longitudinal joint near wheel path Inadequate slab thickness Inadequate load transfer Lack of edge support Panel movement	Proper slab dimensions Adequate overlay thickness Load transfer devices when thickness is greater than or equal to 7 in.	Structural fibers may increase load transfer in thin overlays HMA separation layer should be resistant to stripping	Place shoulder integral with mainline to improve edge support Control the temperature and moisture of the HMA surface at time of paving Adequate curing Saw cut longitudinal joints to T/3	Maintain joint sealant/filler
Transverse Reflective Cracking	Inadequate separation layer Failure to match existing expansion joint locations	HMA separation layer should be a minimum of 1 in. thick	N/A	Match expansion joints in the underlying pavement	N/A
Irregular Reflective Cracking	Wrinkled geotextile fabric	N/A	N/A	Adequate anchoring of the geotextile separation layer	N/A

UBCOC Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Mid-Panel Cracking	Loss of support due to poor drainage and subsequent stripping of HMA separation layer Inadequate overlay thickness and/or wheel overloading HMA separation layer susceptible to stripping	Positive drainage Sealed joints Appropriate slab dimensions	HMA separation layer resistant to stripping	Avoid crushing the underdrain system during construction	Cleaning of the underdrain system
Transverse Cracking	Misaligned dowels due to movement of the dowel basket Inappropriate slab dimensions Structural inadequacy resulting in premature fatigue Differential movement between the UBCOC and underlying pavement	Adequate thickness of HMA separation layer Specify that shipping wires should not be cut	N/A	Securely anchor baskets Adjust anchoring for nonuniform conditions Do not cut shipping wires on the baskets Utilize a dowel bar inserter Saw cut transverse joints to T/3	N/A

UBCOC Distress	Contributing Causes	Prevention: Design	Prevention: Material Selection	Prevention: Construction	Prevention: Maintenance
Transverse Joint Faulting	Deformation of the HMA separation layer under repeated loading Slab movement Erosion of the asphalt binder and fines underneath the joint Panel movement	Load transfer devices when thickness is greater than 7 in. Confirm stability of the HMA prior to final design	Structural fibers may increase load transfer in thin overlays	N/A	Maintain joint sealant/filler
Blowups	Undeployed joints in the UBCOC Panel movement Nonuniform subbase friction	Appropriate slab dimensions for the design thickness Specify sealing joints to prevent incompressibles from filing the joints Provide expansion joints at appropriate intervals. A full-depth transverse saw cut every 12 ft on roadways with minimal truck traffic may reduce the potential for blowups Ensure that HMA separation layer is well drained	N/A	Saw cut joints to T/3 Minimize cold weather paving for UBCOC pavements less than 7 in. thick Seal all joints	Possible mitigation through full depth transverse saw cuts every 300 ft

6. Treatment and Repairs

Maintenance and repair of the distresses discussed in this chapter consist of common procedures. Best practices for these methods can be found in the *Concrete Pavement Preservation Guide* (Smith et al. 2014).

The applicability of these treatments is summarized in Table 18.4.

Table 18.4 Treatment options for UBCOC-specific distresses

Distress	Treatment: Underdrain Maintenance	Treatment: Resealing Joints	Treatment: Diamond Grinding	Treatment: Rout and Seal Cracks/Joints	Treatment: Full-Depth Patching	Do Nothing
Longitudinal Cracking in Tied Shoulders and Widened Sections	✓			✓		✓
Longitudinal Cracking in Wheel Path		✓		✓	✓	✓
Corner Cracking		✓		✓	✓	✓
Transverse Reflective Cracking				✓	✓	✓
Irregular Reflective Cracking				✓	✓	✓
Mid-Panel Cracking	✓	✓		✓	✓	✓
Transverse Cracking				✓	✓	✓
Transverse Joint Faulting	✓	✓	✓			✓
Blowups	✓	✓			✓	

7. References

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CHAPTER 19. FIELD EVALUATION AND LABORATORY TESTING PROCEDURES

1. Description

Field and laboratory testing is an important component in determining the cause of observed distresses. Field evaluation and laboratory testing is summarized in each chapter that pertains to a specific pavement distress. Each chapter then refers the reader to this chapter for a comprehensive analysis of a particular test or procedure.

For each test, the scope of work performed depends upon the type and extent of distresses present, and the relative importance of the project in terms of available budget. In many cases, only a minor amount of testing is required but in some cases, more detailed testing is necessary. The following lists are not all-inclusive but represent some of the most common field and laboratory testing procedures.

2. Field Evaluation Testing Procedures

The following field evaluation procedures may aid in determining the type, extent, and severity of various types of distresses in concrete pavements. The initial step in all project evaluation is a visual survey. Typically, field survey methods are used but for larger projects or an overall network pavement assessment, the visual survey may be done through automated data collection.

Following a thorough analysis of distress mapping and data, destructive or nondestructive test methods provide data on pavement support, materials quality and properties, layer thicknesses, and so on. The most common evaluation procedure involves coring and materials sampling. This procedure provides the material samples required for more detailed laboratory analyses as well as giving an approximation of material properties and condition through visual inspection. It is a good policy to have geotextile/concrete testing performed when material properties are questionable. Table 19.1 summarizes the common field tests for concrete materials.

Table 19.1 Common field tests for concrete materials

Primary Purpose of Test	Distress Associated with Test	Test Designation	Test Name
Determines the level of support beneath a slab	Cracking, settlements, heaves, voids, etc.	ASTM D4694	Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device(FWD); see other tests described below
Determines base and subgrade strength and deformation	Cracking, settlements, and heaves	ASTM D6951	Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications
Determines the modulus of subgrade reaction (static k value)	Subgrade support	ASTM D1195	Standard Test Method for Repetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements
Obtains concrete samples from pavement for testing	All distress types	ASTM C42/C42M – 18	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
Determines pavement layer thickness, location of embedded steel, and detects voids	Cracking, improperly placed embedded steel, pavement voids	ASTM D4748	Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar(GPR)
Determines pavement thickness but must include metal target disks placed on subbase prior to paving	Cracking	N/A	Magnetic Imaging Tomography (MIT Scan-T2)
Determines pavement thickness	Cracking	ASTM C1383-15	Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method

Primary Purpose of Test	Distress Associated with Test	Test Designation	Test Name
Determines voids, cracks under the saw joint, and areas of debonding between layers	Cracking and delamination	N/A	Ultrasonic Tomography (MIRA)
Determines delamination or unsound concrete	Delamination	ASTM D4580/4580M - 12	Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding (which can also be used for road pavements)
Determines dowel bar alignment	Cracking from misaligned dowel bars	ASTM E3013/E3013M	Standard Test Method for Evaluating Concrete Pavement Dowel Bar Alignment Using Magnetic Pulse Induction (Magnetic Imaging Tomography MIT-SCAN2-BT)
Measures faulting at joints	Faulting	AASHTO R36	Standard Practice for Evaluating Faulting of Concrete Pavements
Determines ride quality information and faulting at joints and cracks	Faulting, settlements, and heaves	FHWA-HRT-14-092	Long-Term Pavement Performance Automated Faulting Measurement Using High-Speed Inertial Profilers (HSP)
Estimates the in-place concrete strength	Cracked slabs	ASTM C805	Standard Test Method for Rebound Number of Hardened Concrete
Determines the in-place concrete strength and uniformity through correlation to pulse velocity	Cracking and shattered slabs	ASTM C597	Standard Test Method for Pulse Velocity Through Concrete

Visual Survey

A visual survey is the first step in assessing the type, severity, and extent of pavement distress. Initial visual surveys help to determine what field or laboratory tests to administer. Visual surveys are generally comprised of a number of steps.

First, a driving survey at or near the posted speed limit is used to get an overall assessment of the roadway. Ride quality and obvious distresses determine the general condition of the pavement.

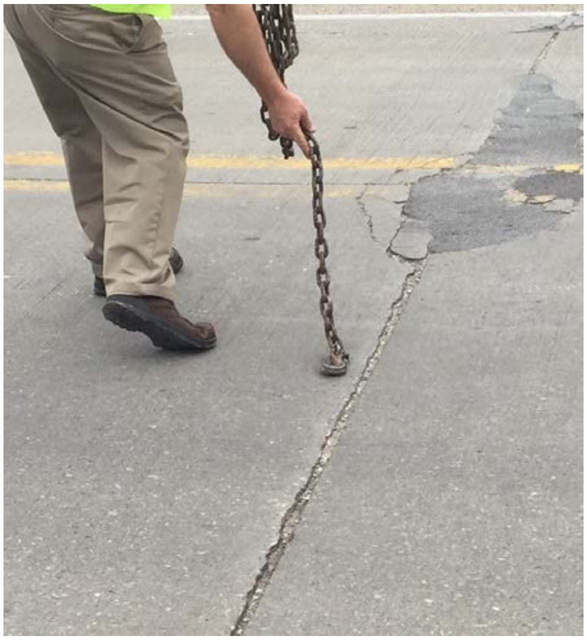
Next, a walking survey may be warranted to map the type, severity, and extent of each noted distress type. In addition to the measuring the visible distresses, probable cause(s) are noted for reference when determining the most feasible repair techniques.

Lastly, a drainage assessment determines if moisture intrusion into the pavement structure is a contributing factor to the distress. Pay particular attention to the functioning of the drainage outlets if a drainage system is present.

Chain Drag or Hammer Sounding – ASTM D4580

Chain dragging determines delamination or unsound concrete. An experienced technician performs chain dragging by dragging a chain and listening to the resulting sound (see Figure 19.1). Similarly, for hammer sounding, an experienced technician strikes the pavement with a hammer and listens to the resulting sound. The basic premise is if the resulting sound from the drag or sounding instrument is a metallic ring, the concrete is essentially sound. If the sound is muffled or dull, there is a high probability of unsound concrete or localized delamination. Pavement deemed “unsound” should be mapped. If large areas of pavement are unsound, evaluate the pavement for repair.

Figure 19.1 Chain drag sounding



Snyder & Associates, Inc.

Falling Weight Deflectometer (FWD) – ASTM D4694

The FWD assesses the level of support under the slabs and determines differences in support (see Figure 19.2). The FWD emits a load pulse to the pavement through a load plate that is 11.8 inches in diameter. This pulse creates a deflection basin and force. Based on this force, various back calculation methods are used to determine the stiffness and uniformity of both the subgrade and base support.

An additional use of the FWD includes determining the load transfer efficiency or degree of interlock between slabs that are adjacent to one another. Load pulse data is taken at two locations an equal distance from either side of the joint. It is typical that loaded slabs show more deflection than unloaded slabs. Ideally, if the joint is working perfectly the deflections on either slab should be equal.

When utilizing the FWD, it is important to note the temperature and surrounding conditions. At cooler conditions, pavement may exhibit alternative results than at hotter temperatures. Concrete slabs have the tendency to warp, expand, or shrink when subjected to temperatures varying from the top of the surface to the bottom of the surface.

FWD testing can also determine the potential for voids beneath the slab. When void detection is performed with the FWD, the deflection is measured at three different loads (9,000, 12,000, and 16,000 pounds). Once the test is complete, deflection versus load is plotted graphically. If the deflection at 0 is larger than 3 mils, then the likelihood that voids are present at that location is high. Alternatively, ground penetrating radar (GPR) may also determine voids beneath the slabs.

Figure 19.2 Falling Weight Deflectometer (FWD)

National Concrete Pavement Technology Center

Plate Load Test – ASTM D1195

When assessing the support conditions of in situ pavement foundation, plate load testing is common. In the past, the static plate load test was a costly, arduous, and tedious test method; however, with the development of the automated plate load test (APLT), the process is much quicker and safer. (See Figure 19.3.) The APLT has the ability to administer several tests to evaluate pavement foundations, stabilized materials, embankments, and compacted fill—some of which are listed below.

- Modulus of subgrade reaction
- Confining stress-dependent resilient modulus
- Bearing capacity test
- Proof wheel rutting
- Borehole shear test
- Tube sampling and extrusion
- Resilient modulus (in situ)
- Strain modulus
- Shear wave velocity
- Cone penetration test
- Rapid air permeability test (in situ)

These advances in technology allow for testing of the resilient modulus in the field, whereas previously, the modulus could only be tested in the laboratory.

The plate load test consists of a loading device that has support points that are at least 8 feet apart, a hydraulic jack assembly for load application, a bearing plate of 6 to 30 inches in diameter and at least 1-inch thick, and a deflection beam that includes three or more dial gauges that record deflection. The testing area shall be at least twice the diameter of the chosen bearing plate. Loading is incrementally administered until total deflection has been reached. At each increment, the load and deflection is recorded. An undisturbed sample is also acquired for administration of laboratory testing.

Figure 19.3 Automated plate load test

Ingios Geotechnics, Inc.

Dynamic Cone Penetrometer (DCP) – ASTM D6951

The DCP measures the base course, subgrade strength, and deformation properties of the pavement and subgrade. The concept behind the DCP test is that the resistance to the penetration of solid objects is directly correlated to the strength of the soil. This test is both rapid and inexpensive. It evaluates support conditions to a depth of approximately 4 feet (1.2 m). It is necessary to perform the test after core extraction but prior to removing the material samples. The DCP test uses a 16.7-pound drop hammer that falls along the penetrometer shaft to drive a cone attached to a steel rod into the pavement or subgrade. The hammer is dropped from a fixed height of 22.6 inches. There are versions the DCP that utilize a 10-pound hammer (weaker soils) or possess a disposable cone. The penetration rate of each drop is recorded. Following the test, the average is calculated at the layer midpoint (the midpoint minus 2 inches) and at the midpoint plus 2 inches. These calculations identify layer boundaries and the California bearing ratio (CBR) of each layer. See Figure 19.4a for a handheld DCP and Figure 19.4b for trailer-mounted DCP.

Figure 19.4 Dynamic cone penetrometer (DCP)

a. Handheld DCP
Institute for Transportation, Iowa State University

b. Trailer-mounted DCP
Province of Quebec, Canada

Coring and Material Sampling – ASTM C42/C42M – 18

Laboratory testing is key in determining relevant material properties. In subgrade soil cores, the soil classification (AASHTO), gradation, in situ moisture, density, sulfate resistance, and soil stiffness (resilient modulus) can be determined. In concrete pavement cores, the layer type, layer thickness, a visual review of the core, the distress cause, and the need for petrographic analysis can be discovered. See Figure 19.5.

The most common method of field sampling is coring the pavement and extracting material samples of both the base (if present) and subgrade soil. A rotary core drill is used, accompanied by a hollow cylindrical barrel (perpendicular to the pavement), to cut into the pavement and obtain the sample for testing. Common core diameters are 2, 4, and 6 inches. If aggregate size is larger than 1.5 inches or petrographic testing is requested, the use of a 6-inch core is recommended. To obtain samples of the soil, subbase, or base, administer other specialized testing at the core hole. During laboratory evaluation of the core, it is critical to maintain the in situ moisture content of the sample. When evaluating settlement, determination of the in-place density may provide useful information.

Figure 19.5 Concrete pavement coring

Global Pavement Consultants, Inc.

Ground Penetrating Radar (GPR) – ASTM D4748

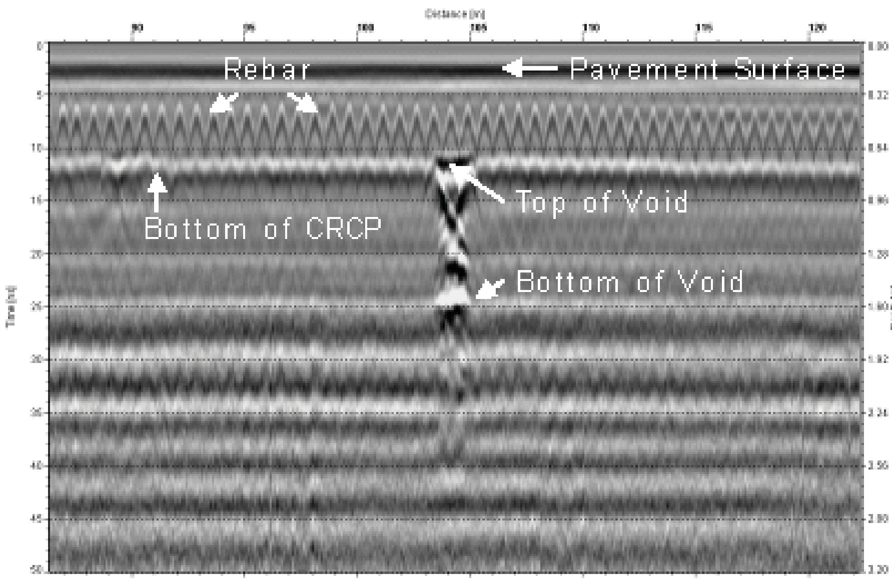
GPR uses the amplitude and time of a radio pulse to analyze the slab thickness, location of embedded steel, detection of voids under the slab, location of defects, or changes in the overall pavement structure. GPR emits radio waves that are reflected and recorded at different testing locations to be used for a variety of purposes in pavement evaluation. GPR uses radar equipment consisting of an antenna, control unit, data collection, computer, and software. The antenna can be either air coupled or ground coupled, referring to the location of the antenna relative to the pavement surface. The air-coupled configuration can be used at higher speeds (see Figures 19.6 and 19.7), but it is less able to distinguish between certain materials. The ground-coupled configuration provides a better signal penetration into the ground, but it is limited to slower test speeds because of its contact with the pavement surface.

Longer duration of reflected wave pulses signifies thicker pavement layers, while more substantial wave pulse amplitudes indicate higher moisture content. GPR is an appreciable tool to utilize when determining the location of embedded steel as the wave pulses reflect metal completely.

Figure 19.6 Ground penetrating radar (GPR)

Maser et al. 2010

Figure 19.7 GPR scan of CRCP



Chen 2010, Texas DOT

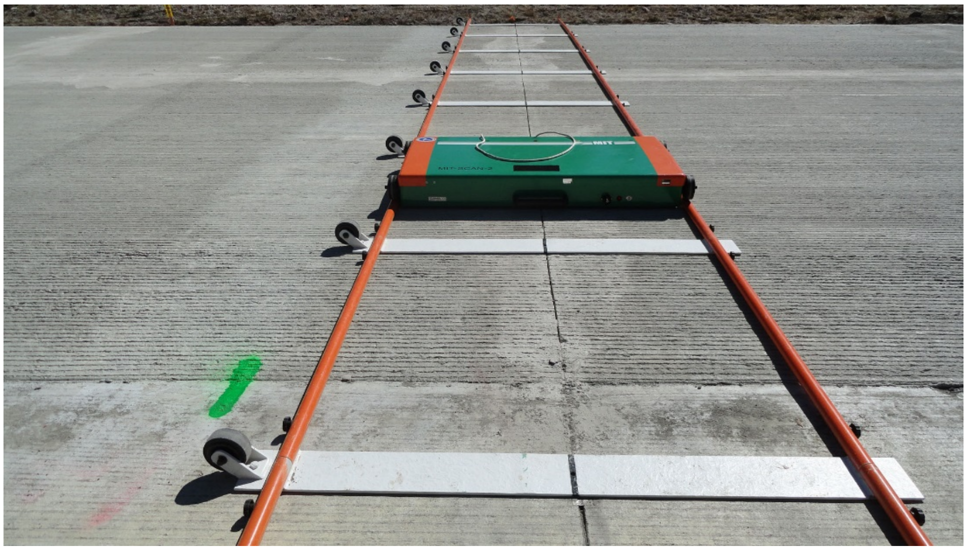
Magnetic Imaging Tomography (MIT Scan-2 and MIT-SCAN-T2 Devices)

MIT emits an electromagnetic pulse and detects the induced magnetic field (Yu and Tayabji 2007) MIT scanning accurately and nondestructively determines the location of either dowel bars or tie bars in the pavement, and determines the pavement thickness.

MIT Scan-2 is specifically designed to analyze the dowel bar depth, side shift, and horizontal and vertical misalignments. (See Figures 19.8 and 19.9.) This test provides the most accurate method of determining the placement and alignment of the embedded steel but can only be used for discrete steel elements. It does not provide accurate results if the shipping wires on dowel baskets are not cut prior to paving. The MIT Scan-2 consists of three components: a sensor unit, an onboard computer, and a glass fiber reinforced rail system. Electromagnetic pulses are sent out from the sensor unit as it glides along the rail system over the joint and identifies the induced magnetic field. Data that is collected during the testing is stored in the memory card equipped in the onboard computer. The MIT Scan-2 has the ability to withstand rain, dust, and even low temperatures, which is why it is widely used in the construction field.

MIT-SCAN-T2 is typically used to determine slab thickness but requires reflective disks to be placed on the base surface prior to paving. The T2 uses pulse induction technology to detect the reflective disks, and then determines the pavement thickness nondestructively. The T2 test provides accurate, quality testing at lower costs, faster measurements, and at various locations without having to cut cores. As with the MIT Scan-2, the T2 has many acceptable features that make it easy to use in the construction field.

Figure 19.8 MIT Scan-2 device



FHWA Mobile Concrete Laboratory (MCL) 2011

Figure 19.9 MIT-SCAN-T2 device in use

Yu and Tayabji 2009

Ultrasonic Tomography (MIRA Device)

MIRA is used to determine pavement thickness and relative concrete strength, locate embedded steel, identify full- or partial-depth cracking or areas of debonding, joint deterioration, and poor consolidation through impact-echo technology. Advantages to using MIRA for concrete inspection are that it provides a detailed visualization of the concrete interior, it is very efficient and easy to use, it is nondestructive, it is precise, and no preparation is needed.

The measuring block, which is placed on the surface, utilizes 48 transducers at an operating frequency of 50 kHz in combination with synthetic aperture focusing and combinational sounding to collect and process data (see Figure 19.10). MIRA is equipped with a screen to access preliminary information, which is displayed with different colors. MIRA has various operating modes, and the ability to test various objects, making it a versatile and effortless device to utilize.

Figure 19.10 MIRA

National Concrete Pavement Technology Center 2013

Faulting – AASHTO R36

Assessing the amount of faulting on a section of pavement is necessary because faulting can be the cause of many underlying issues such as settlement and heaving. While faults can be measured manually, the Georgia Fault Meter measures faulting at joints quickly and electronically. The meter has a push button on they handle that provides a digital readout when pressed. The unit must be set so that the legs of the fault meter are in the direction of the traffic on the leave side of the joint (Figure 19.11). Record the faulting to the nearest millimeter at a location 9 feet (2.75 m) from the roadway centerline. At each location, take three measurements and record the average of the readings. If the approach slab is higher than the departure slab, the reading is recorded as positive (+); if the approach slab is lower, faulting is recorded as negative (-).

Figure 19.11 Georgia Faultmeter

Miller and Bellinger 2014

High-Speed Inertial Profiler

For use of the high-speed profiler (HSP), follow the procedures described in *Long-Term Pavement Performance Automated Faulting Measurement* (Agurla and Lin 2015). HSPs have accelerometers that measure the movement of the vehicle frames and sensors (typically lasers) that record the displacement between the vehicle and the pavement at certain fixed intervals. Together these devices provide a longitudinal profile along the wheel paths and ride quality information, including faulting at joints and cracks. Since the HSP operates at highway speed, the advantage of this device is faster data collection without the need for traffic control, lane closures, safety measures, and inherent costs. (See Figure 19.12.)

Figure 19.12 High-speed pavement profiler

©Ames Engineering

Rebound Number of Hardened Concrete – ASTM C805

This test determines the rebound number of concrete to evaluate cracked slabs by the use of a spring-driven hammer equipped with a steel plunger (see Figure 19.13). The plunger is held perpendicular to the specimen's surface and the instrument is pressed until the plunger on the hammer impacts the specimen. The rebound number is recorded 10 times for each specimen. By determining the rebound number, areas of inadequate concrete and estimated strength are established.

Figure 19.13 Rebound hammer

Tumcivil.com

Ultrasonic Pulse Velocity through Concrete – ASTM C597

Ultrasonic pulse velocity is utilized to determine the presence of voids or cracks within the concrete. This test determines the wave's pulses of propagation velocity (of longitudinal stress) through concrete. The surface of concrete in question is put in contact with an electro-acoustical transducer that emits pulses of longitudinal stress. The ultrasonic pulses are converted into electrical energy by a secondary transducer displaced at a certain distance from the transmitter (see Figure 19.14). Based on the transducer distance and time the pulse velocity is calculated. This information can aid in determination of suitable repairs.

Figure 19.14 Ultrasonic pulse velocity test system

TechRentals 2016, https://www.youtube.com/watch?v=a-nAT73bgvA

3. Laboratory Testing Procedures

Conduct laboratory testing in conjunction with a comprehensive field evaluation to determine specific material properties or behavior. The extent of the testing program depends primarily on the types of observed distresses and the available funds. The tests shown below, in Table 19.2, are differentiated by concrete material tests and those used to characterize the base and subgrade materials. This list is not all-inclusive; note that new test methods and equipment may be available to address a specific testing need. The descriptions of each test listed below are brief and the actual testing guide should be referenced and followed when administering the testing procedure.

Table 19.2 Common laboratory tests for subgrade and/or base materials

Primary Purpose of Test	Typical Distress Associated with Test	Test Designation	Test Name
Determining the overall gradation of the support layers	Settlement and heaves, cracking, shattered slabs, faulting	AASHTO T 27, ASTM C136	Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates
Determining the amount of fines present primarily in an unbound base course	Settlement and heaves, cracking, shattered slabs, faulting	AASHTO T 11, ASTM C117	Standard Method of Test for Materials Finer Than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing
Determining the fines fraction and characteristics of the base course aggregate and the overall particle size distribution of the subgrade soil	Settlement and heaves, cracking, shattered slabs, faulting	AASHTO T 88	Standard Method of Test for Particle Size Analysis of Soils – Hydrometer Test
Determining the plasticity and moisture sensitivity of the fines; commonly referred to as the Atterberg limits	Settlement and heaves, cracking, shattered slabs, faulting	AASHTO T 90	Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils
Determining the subgrade volumetric changes due to variations in subgrade moisture, frost penetration, and growing ice lenses	Settlement and heaves, cracking, shattered slabs, faulting	AASHTO T 99, AASHTO T 180	Standard Method of Test for Moisture-Density Relations of Soils
Determining the dynamic strength/ deformation characteristics of the material being tested, base, or subgrade	Settlements, cracking, and shattered slabs	AASHTO T 307	Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Minerals

Base and Subgrade Testing

The subgrade and base characterization may be carried out by field evaluations including the FWD, DCP, plate load test, and others. However, specific details regarding the soundness of the aggregates, moisture sensitivity, strength/deformation characteristics, and other important properties may require laboratory testing of field extracted material samples.

It is common practice to first core the pavement to remove a 4- or 6-inch diameter concrete core and then auger or otherwise obtain samples of the support layers. Depending on the type of distress present, and if support conditions are a likely cause, one or more of the tests shown in Table 19.2 may be applicable. At a minimum, the base course gradation, moisture content, thickness, subgrade gradation, Atterberg limits, and moisture content should be determined.

Sieve Analysis of Fine and Coarse Aggregates – ASTM C136/AASHTO T 27

This test determines the particle size distribution of aggregates through the use of sieves to evaluate settlement, heaves, cracking, shattered slabs, and faulting. This information is particularly useful when determining if an aggregate meets specifications. Use of an improper aggregate can cause problems with drainage, consolidation, and many other factors that lead to pavement distress. To determine a sample's particle size distribution, the sample of aggregate (dry) is segregated through several sieves. Sometimes a mechanical shaker is used to vibrate the sample to ensure an adequate distribution. The amount contained on each sieve determines the aggregate's particle size distribution and can prove useful when analyzing porosity and consolidation. (See Figure 19.15.)

Figure 19.15 Sieve analysis

Sepor, Inc. 2018, https://www.sepor.com/product/w-s-tyler-ro-tap-sieve-shaker/

Materials Finer Than the No. 200 Sieve in Mineral Aggregates by Washing – ASTM C117/AASHTO T 11

This test determines the amount of particles within an aggregate that are finer than the No. 200 sieve to evaluate settlement, heaves, cracking, shattered slabs, and faulting. The sample is weighed and then washed in water, therefore passing any dissolved material through the No. 200 sieve. The material lost is the material finer than the No. 200 sieve. If the amount of material finer than the No. 200 sieve is a sizable amount, it could be an indication aggregate degradation. Aggregate degradation leads to inadequate support, which then leads to pavement distress. (See Figure 19.16.)

Figure 19.16 Washing materials through a No. 200 sieve

Iowa DOT

Particle Size Analysis of Soils – Hydrometer Test – AASHTO T 88

This test is an additional test to determine the particle size distribution of an aggregate to evaluate settlement, heaves, cracking, shattered slabs, and faulting. A sieve analysis is performed on the sample before it goes through dispersion (see Figure 19.17). A graduated hydrometer is immersed in the dispersion specimen. Hydrometer readings (at 2, 5, 30, 60, 250, and 1440 minutes) are taken during this test determine the percentage of silt and clay in the aggregate. This aids in determining the soil type. As mentioned above, if the finer material amount is large, it could be an indication aggregate degradation, which is a problem if the aggregate is to be used as support. Inadequate support leads to pavement distress.

Figure 19.17 Hydrometer test

FHWA, Highway Materials Engineering Course

Determining the Plastic Limit and Plasticity Index of Soils – AASHTO T 90

This test (often referred to as the Atterberg limits) is used to determine the plasticity of a soil to evaluate settlement, heaves, cracking, shattered slabs, and faulting. The plasticity of a soil is when the moisture content of the soil is in a plastic state. A sieve analysis is based on the requirements of ASTM C136 and the material retained on the No. 40 sieve is kept and used as the sample for this test. The air-dried sample is mixed with tap water until it can be formed into a ball. The sample is then subjected to a rolling procedure. Finally, the plasticity index is calculated. Knowing the plastic limits of a solid helps determine the moisture sensitivity of the fines. Soils with high moisture sensitivity can aid in creation of pavement distresses. (See Figure 19.18.)

Figure 19.18 Atterberg limit test

FHWA, Highway Materials Engineering Course

Moisture-Density Relations of Soils

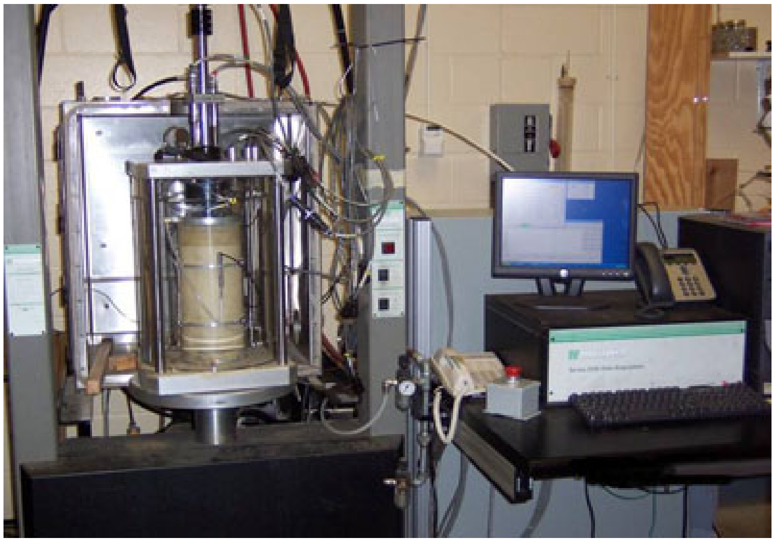
This procedure (sometimes referred to as the Proctor test) covers the determination of the moisture-density relations of soils and soil-aggregate mixtures in accordance with two similar test methods: AASHTO T 99-17 and AASHTO T 180-17. Both use Methods A, B, C, and D.

The tests are for the design and control of the compaction of soils to determine the required degree of compaction and a method to obtain that compaction. These test methods are based on soils compacted in a mold of a given size with a rammer dropped from a given height.

Determining the Resilient Modulus of Soils and Aggregate Minerals – AASHTO T 307

This testing procedure determines the resilient modulus of a soil or aggregate material to evaluate cracking and shattered slabs. The sample (6-inch core) is loaded into a triaxial chamber device and is subjected to loading; see Figure 19.19. Transducers measure the deformation during different load applications. This testing allows the resilient modulus to be calculated from the deviator stress and axial strain values. The resilient modulus evaluates the dynamic strength/deformation characteristics of the material being tested.

Figure 19.19 Resilient modulus test



Minnesota DOT

Concrete Testing

Concrete testing is typically performed to verify strength, consolidation, uniformity, segregation, hardened air content, aggregate materials reactivity or issues (alkali silica reaction [ASR], alkali carbonate reaction [ACR], D-cracking), chloride permeability, and potentially others. The majority of these tests are specified by either AASHTO or ASTM. The listed protocols have the appropriate AASHTO or ASTM test designation, which should be consulted to determine their applicability to specific distresses. See Table 19.3.

Table 19.3 Common laboratory tests for concrete materials

Primary Purpose of Test	Distress Associated with Test	Test Designation	Test Name
Determining the compressive strength of concrete	Cracking and shattered slabs	AASHTO T 22, ASTM C39	Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens
Determining the flexural strength of concrete (preferred method)	Cracking and shattered slabs	AASHTO T 97, ASTM C78	Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
Determining the flexural strength of concrete	Cracking and shattered slabs	AASHTO T 177, ASTM C293	Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading)
Determining the entrained and entrapped air content as well as the structure of the air	Consolidation, uniformity, freeze thaw damage, spalling, scaling	ASTM C457	Standard Test Method for Determination of Parameters of the Air-Void System in Hardened Concrete
Determining the freeze/thaw resistance of concrete	Freeze thaw damage, spalling, scaling, delamination	ASTM C666/C666M – 15, AASHTO T 161	Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing
Determining the scaling resistance of concrete when deicing chemicals are used	Scaling and spalling	ASTM C672/C672M	Standard Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals
Determining the elastic modulus (Young's modulus) of the concrete (not frequently performed)	Cracking and shattered slabs	ASTM C469	Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
Determining the resistance to polishing of the concrete due to tire wear	Surface polishing and loss of friction	ASTM C779/C779M	Standard Test Method for Abrasion Resistance of Horizontal Concrete Surfaces

Primary Purpose of Test	Distress Associated with Test	Test Designation	Test Name
Determining the ease with which chloride ions from deicing salts can be absorbed into the concrete (preferred method)	Spalling and delamination	AASHTO TP 95	Standard Method of Test for Surface Resistivity Indication of Concrete's Ability to Resist Chloride Ion Penetration
Determining the ease with which chloride ions from deicing salts can be absorbed into the concrete	Spalling and delamination	AASHTO T 277, ASTM C1202	Standard Method of Test for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
Determining the ease with which chloride ions from deicing salts can be absorbed into the concrete	Spalling and delamination	AASHTO T 259	Standard Method of Test for Resistance of Concrete to Chloride Ion
Determining if alkali-silica or alkali-carbonate reactions have occurred and to what extent	Cracking, spalling, scaling, delamination, and other materials-related distresses	ASTM C856 – 18a	Standard Practice for Petrographic Examination of Hardened Concrete
Determining the expansion characteristics of concrete as a function of temperature	Blowups, compression spalls, and delamination	AASHTO T 336	Standard Method of Test for Coefficient of Thermal Expansion of Hydraulic Cement Concrete

Compressive Strength of Cylindrical Specimens - ASTM C39/AASHTO T 22

The compressive strength of concrete is useful to evaluate cracked or shattered slabs. Typically, concrete strength requirements are specified based on compressive or flexural strength. Prior to allowing traffic on the roadway, the specified strength must be met. To determine the compressive strength of the concrete, inspectors take a sample of the concrete in cylindrical form following standards specified by the project area's jurisdiction. Once the sample is in the lab it is put in a testing machine and a compressive axial load is applied until the cylinder fails (see Figure 19.20). Machines used to test the cylinder should be able to apply standard load rates and be equipped with two bearing blocks. The maximum load achieved during testing reflects the strength of the in situ concrete.

Figure 19.20 Concrete compression testing

Graybeal 2006

Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) – ASTM C78/AASHTO T 97

This test method is an alternative to ASTM C39 (listed above) but instead of using a cylinder for testing this test implements a simple concrete beam. Once the sample is acquired, it is loaded into an apparatus that utilizes third-point loading and bearing blocks to test the flexural strength of the concrete (see Figure 19.21). The load is applied continuously at a constant, increasing rate until the beam breaks. Upon breaking, the modulus of rupture or flexural strength of the specimen is calculated.

Figure 19.21 Concrete beam testing, third-point loading

Todd Hansen, Iowa DOT

Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading) – ASTM C293/AASHTO T 177

This test is similar to that of the ASTM C78 but is able to produce higher values of flexural strength. This test procedure is useful to evaluate cracked or shattered slabs. The load is applied at the center point of the beam and perpendicular to the face of the beam. The specimen is loaded continuously at a constant rate between 125 and 175 psi/minute until the beam reaches its breaking point. Upon breaking, the modulus of rupture or flexural strength of the beam can be calculated. See Figure 19.22.

Figure 19.22 Concrete beam central (single) point loading test

Todd Hansen, Iowa DOT

Scaling Resistance of Concrete Surface Exposed to Deicing Chemicals – ASTM C672

This test accesses the concrete's ability to withstand scaling during freeze-thaw cycles while deicing chemicals are present. Specimens to be tested must have a surface area of 72 square inches and be at least 3 inches in depth. If surface treatments or curing is assessed during this test, they should be applied to the sample. The specimens are then subjected to a freezing (0 ± 5 °F) environment for a duration of 16 to 18 hours. Next, the specimens are subjected to a warm, (73.5 ± 3.5 °F) humid (45 to 55%) room for 6 to 8 hours. This cycle is repeated 50 times. At the end of the testing period, the surface condition is visually evaluated.

Hardened Air Petrographic Analysis (Microscopial Determination of Parameters of the Air Void System in Hardened Concrete) – ASTM C457

This test provides the ability to test the air content of concrete that has already hardened to evaluate freeze-thaw damage and scaling. Several problems arise when the air content of the concrete is inadequate. Air behind the paver should be at least 5%. This procedure is a preferred choice when freeze-thaw is suspected. The sample is typically a core of 3- to 4-inch diameter and 4 to 6 inches long (see Figure 19.23). The results of these tests include percent of total air, specific surface area, paste-air ratio, spacing factor, number of voids per inch, and percent of paste.

Figure 19.23 Air void testing apparatus

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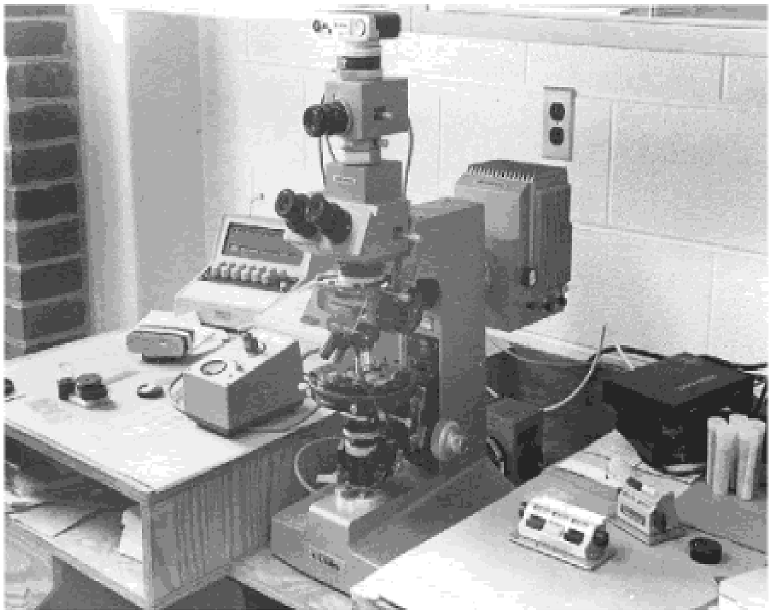
Petrographic Examination of Aggregates for Concrete – ASTM C295/C295M

This test examines the petrographic properties of aggregates proposed for use in a concrete mixture or for use as a raw material. This test procedure is useful to evaluate cracking, spalling, scaling, delamination, and materials distresses. Much of this examination is completed with the use of optical microscopy (Figure 19.24), differential thermal analysis, scanning electron microscopy (Figure 19.25), x-ray diffraction, infrared spectroscopy, and energy dispersive x-ray analysis.

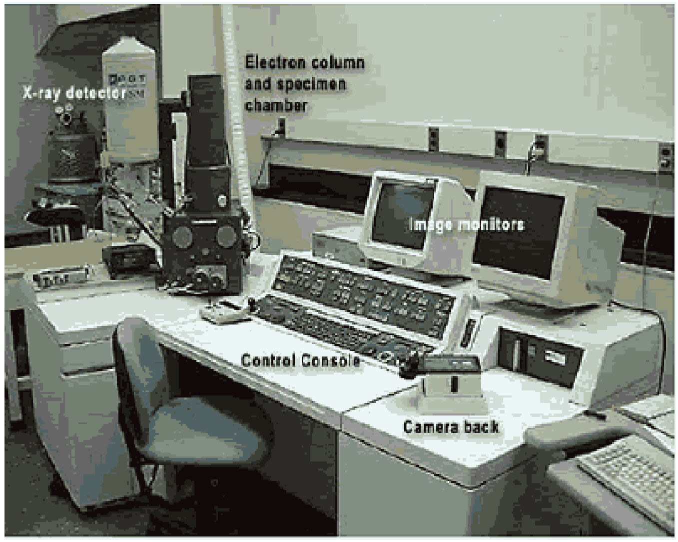
Samples are selected and acquired based on the stipulations of random sampling of aggregates. Samples are then run through sieve analysis and the amount retained in each sieve is recorded. A petrographer then examines the sample with a petrographic microscope or other eligible testing apparatus. Samples taken as a drilled core should follow ASTM C42 and the materials types within should be recorded. All petrographic, chemical, physical, or geological investigations are analyzed and recorded. Lastly, an educated decision of acceptability of the aggregate based on all findings is made. This test aids in determining if alkali-silica or alkali-carbonate reactions have occurred and to what extent.

Figure 19.24 Petrographic microscope



FHWA

Figure 19.25 Scanning electron microscope



FHWA

Permeability – ASTM C1202/AASHTO T 277

The so-called "rapid chlorine" test (see Figure 19.26) is currently the most commonly used "permeability" measurement. The test does not measure permeability directly but measures conductivity, which is a surrogate test for permeability. It is primarily intended for use in reinforced elements. The data is used for comparison purposes between concrete mixtures.

While intended for use at the design stage, numerous authorities are using the test for acceptance purposes. A minimum value of 1,500 coulombs at 56 days is an indicator of more than adequate performance for pavements.

Figure 19.26 Surface resistivity test equipment

FHWA

Resistance of Concrete to Rapid Freezing and Thawing – ASTM C666/C666M – 15/AASHTO T 161

This test determines the resistance of concrete to freeze-thaw cycles. Specimens consist of cores between 3 and 5 inches in diameter, 11 to 16 inches in length, and that comply with AASHTO M 210 and R 39. The specimens cure for 14 days before being subjected to the testing procedure. During testing, the specimens are first brought to the target thaw temperature by being placed in thawing water. The specimen is measured and then placed in the freezing apparatus (see Figure 19.27). This is repeated 300 times or until the dynamic modulus reaches 60% of its initial value. Measurements, defects (and their corresponding cycle), and loss or gain of mass is evaluated and recorded.

Figure 19.27 Freeze-thaw chamber

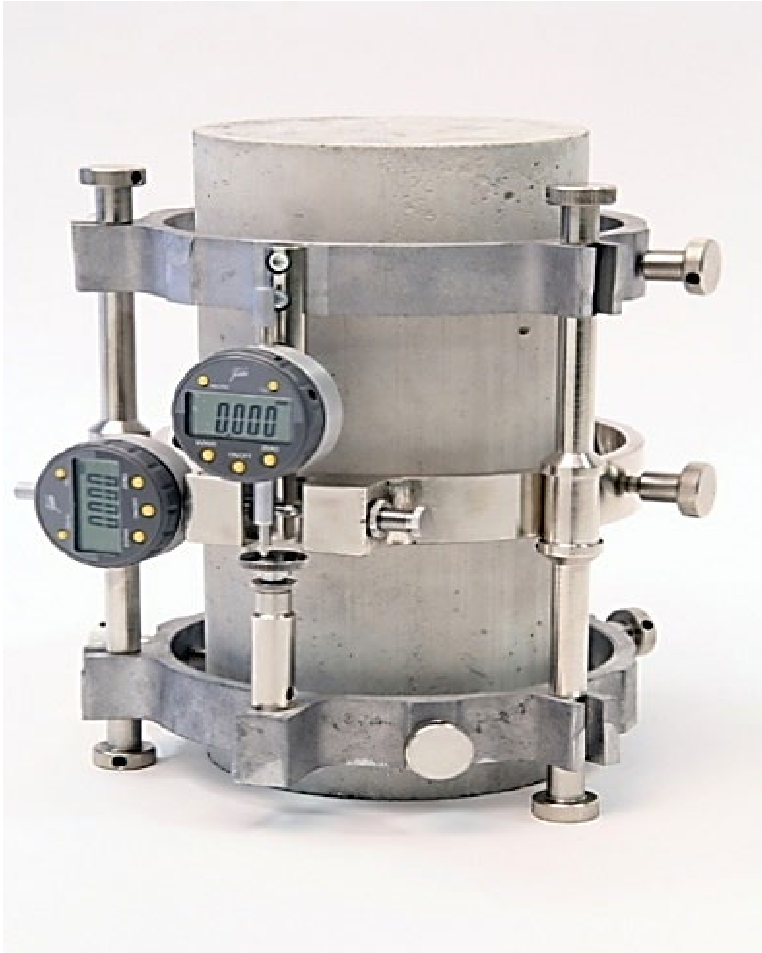
FHWA

Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression – ASTM C469

This test can provide results at any age of concrete under any condition, which makes it a suitable candidate for various pavements. This test determines the modulus of elasticity and Poisson's ratio. Acceptable test specimens include molded cylindrical specimens complying with ASTM C192 and drilled core specimens complying with ASTM C42.

The specimens are loaded into a testing machine conforming to ASTM C39 testing requirements, with a compressometer attached, to determine the modulus of elasticity (Figure 19.28). Several sensing devices equipped on the compressometer measure the deformation. The specimen is loaded twice and an average of the deflection readings is taken. These readings are then used to calculate the modulus of elasticity and Poisson's ratio.

Figure 19.28 Beam with compressometer



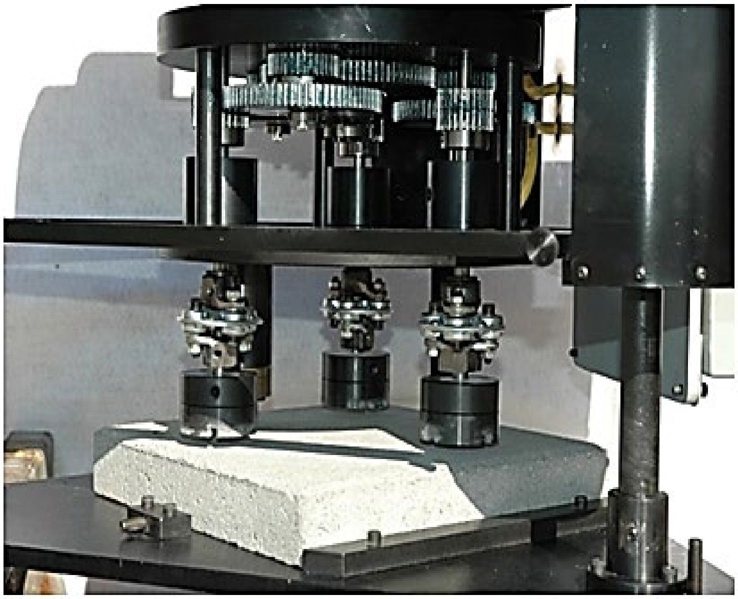
Controls-group.com

Abrasion Resistance of Horizontal Concrete Surfaces – ASTM C779/C779M

This test assesses horizontal concrete's resistance of abrasion to mixture proportions, finishing, and surface treatments. The testing equipment utilized in this test is portable, making it versatile when the locations of the concrete vary significantly. Test specimens are 12 by 12 by 4 inches.

Procedure A of this test implements a revolving free-floating disk machine accompanied by abrasive grit that slides and scuffs (Figure 19.29). The machine drives at 12 rpm transversely in a circular path. While maneuvering on the path, the disks are individually turned on their own axis with a load of 5 lbf on each.

Figure 19.29 Disk machine



Whitemachine.com

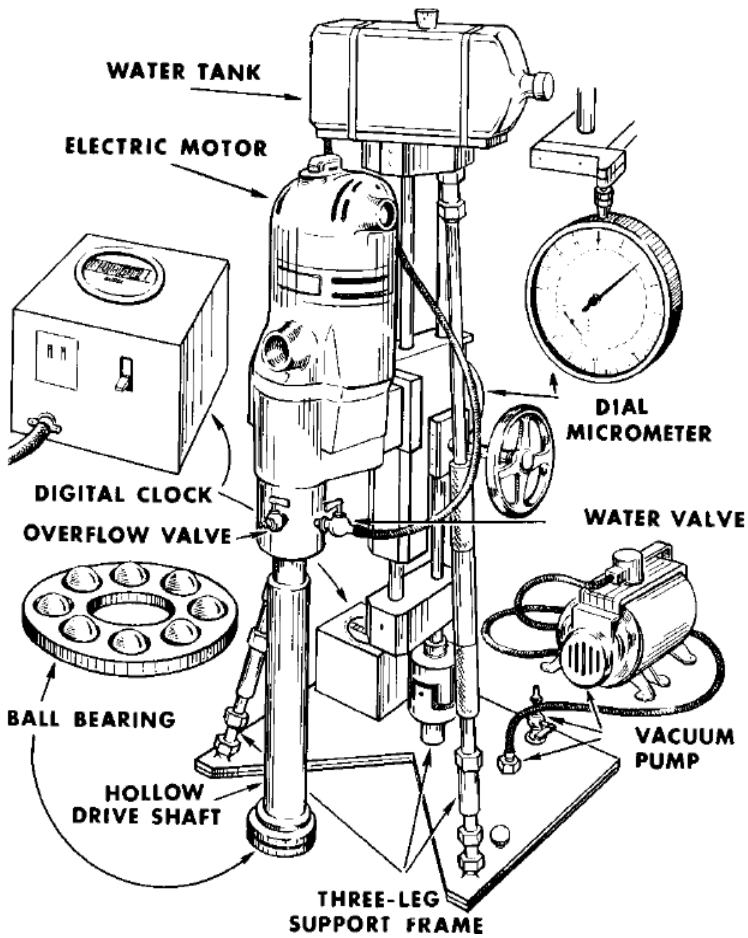
Procedure B of this test utilizes a dressing wheel machine that uses impact and sliding friction of the wheels to test the specimen (Figure 19.30). The dressing wheel machine is equipped with three sets of seven dressing wheels, loaded at 16.5 pounds and powered by a motor at 56 rpm. The machine operates in an abrasive circular path (8.5 inches, outer diameter).

Figure 19.30 Dressing wheel machine

Nox-Crete

Procedure C employs high contact stresses, impact, and sliding friction from a ball bearing machine to access the concrete sample (Figure 19.31). This test employs abrasion to wet concrete by utilization of a quickly motorized rotating ball bearing. The water on the concrete rinses loose particles from the surface, allowing the ball bearing to be in direct contact with the concrete specimen being tested. The load administered to the ball bearing is 27 lbf, which includes the motor, drive shaft, and water. The drive shaft of the ball bearing machine operates at 1,000 rpm. Dial readings are taken every 50 seconds until either 1,200 seconds or an eroded depth of 0.1225 inches is reached.

Figure 19.31 Ball bearing abrasion test machine



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After each procedure, the time passed is plotted versus the depth of wear. These procedures aid in determining if the concrete is abrasion resistant at the surface or at a greater depth.

Coefficient of Thermal Expansion of Hydraulic Cement Concrete - AASHTO T 336

This test determines the coefficient of thermal expansion (CTE) that aids in quantifying to what extent materials change (in length) due to varying temperature. CTE has a huge impact on the opening and closing of joints, and these impacts can lead to pavement distress. A sample is taken according to regulations in ASTM C42. Samples are loaded into the CTE frame (Figure 19.32) and the software runs tests on the sample to determine the CTE.

Figure 19.32 CTE frame

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Standard Practice for Petrographic Examination of Hardened Concrete-ASTM C856-18a

This practice outlines procedures for the petrographic examination of samples of hardened concrete. The samples examined may be taken from concrete constructions, they may be concrete products or portions thereof, or they may be concrete or mortar specimens that have been exposed in natural environments, or to simulated service conditions, or subjected to laboratory tests. The phrase “concrete constructions” is intended to include all sorts of objects, units, or structures that have been built of hydraulic cement concrete.

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