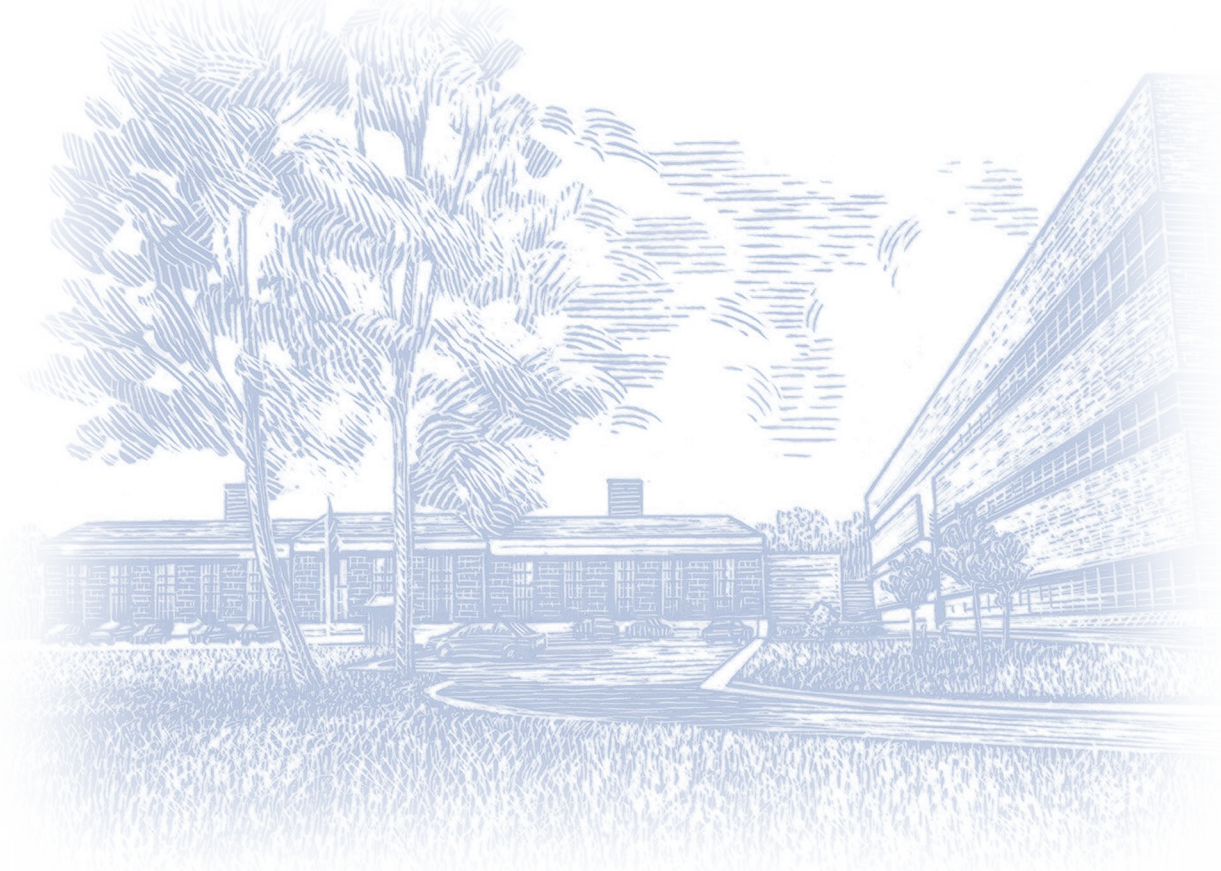


Guidelines for Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements Volume 1

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Foreword

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Executive Summary

Well-designed and constructed portland cement concrete (PCC) pavements are inherently durable and are expected to be relatively maintenance free during many years of service. Primarily because of these two properties—durability and low maintenance—concrete has been the material of choice for premium pavements for many years.

Unfortunately, a number of pavement distresses can occur as a result of the interaction between the concrete and the environment in which it serves. In contrast to the common perception held by many engineers that concrete is relatively inert, it is in fact a very complex material whose properties can change significantly with time. Some of these changes can be positive, such as long-term strength gain obtained through continued cement hydration. Other changes can be detrimental to the concrete, resulting in the development of premature pavement distress. Paste deterioration resulting from freezing and thawing, aggregate freeze-thaw deterioration (also referred to as D-cracking), and alkali-aggregate reactivity (AAR) are a few examples of detrimental changes that can occur over time. When these changes manifest themselves on the pavement surface, they fall under the general category of materials-related distress (MRD). The types of MRD that are common to concrete pavement, and thus were of primary concern in this project, are summarized in Table 1.

MRD in concrete pavement is a concern to all State highway agencies (SHAs) in the United States. Although the specific type of distresses may vary geographically, it is clear that a better understanding of these types of pavement failures is an important starting point for the production of pavements having longer expected service lives. A principal driving force for understanding the sources of MRD is an increased awareness on the part of SHAs that such failures occur. However, the ability of these agencies to analyze, diagnose, remedy, and prevent these failures has not been fully developed, and this research project focuses on these general areas.

The objective of this research was to develop guidelines to provide pavement engineers and field and laboratory personnel with a systematic procedure for the identification, evaluation, treatment, and prevention of MRD in PCC pavements.

This *Volume 1: Final Report* presents the synthesis of background material, describes the development of the guidelines, and briefly introduces the case studies. The other two reports are:

- Volume 2: Guidelines Description and Use.
- Volume 3: Case Studies Using the Guidelines.

Table 1. Summary of key MRDs in concrete pavements.

Type of MRD	Surface Distress Manifestations and Locations	Causes/ Mechanisms	Time of Appearance	Prevention or Reduction
<i>MRD Due to Physical Mechanisms</i>				
Freeze-Thaw Deterioration of Hardened Cement Paste	Scaling, spalling or map-cracking, generally initiating near joints or cracks; possible internal disruption of concrete matrix.	Deterioration of saturated cement paste due to repeated freeze-thaw cycles.	1–5 years	Addition of air-entraining agent to establish protective air void system.
Deicer Scaling/Deterioration	Scaling or crazing of the slab surface with possible alteration of the concrete pore system and/or the hydrated cement paste leading to staining at joints/cracks.	Deicing chemicals can amplify freeze-thaw deterioration and may interact chemically with cement hydration products.	1–5 years	Provide minimum cement content of 335 kg/m ³ , limit water–cement ratio to no more than 0.45, and provide a minimum 30-day "drying" period after curing before allowing the use of deicers.
Freeze-Thaw Deterioration of Aggregate	Cracking parallel to joints and cracks and later spalling; may be accompanied by surface staining.	Freezing and thawing of susceptible coarse aggregates results in fracturing and/or excessive dilation of aggregate.	10–15 years	Use of non-susceptible aggregates or reduction in maximum coarse aggregate size.
<i>MRD Due to Chemical Mechanisms</i>				
Alkali–Silica Reactivity (ASR)	Map cracking over entire slab area and accompanying expansion-related distresses (joint closure, spalling, blowups).	Reaction between alkalis in the pore solution and reactive silica in aggregate resulting in the formation of an expansive gel and the degradation of the aggregate particle.	5–15 years	Use of non-susceptible aggregates, addition of pozzolans to mix, limiting total alkalis in concrete, minimizing exposure to moisture, addition of lithium compounds.
Alkali–Carbonate Reactivity (ACR)	Map cracking over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Expansive reaction between alkalis in pore solution and certain carbonate/dolomitic aggregates which commonly involves dedolomitization and brucite formation.	5–15 years	Avoid susceptible aggregates, significantly limit total alkalis in concrete, blend susceptible aggregate with quality aggregate or reduce size of reactive aggregate.
External Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area, ultimately resulting in	Expansive formation of ettringite that occurs when external sources of sulfate (e.g., groundwater, deicing	1–5 years	Use w/c below 0.45, minimize tricalcium aluminate content in cement, use blended

	joint or surface deterioration.	chemicals) react with the calcium sulfoaluminates.		cements, use pozzolans.
Internal Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Formation of ettringite from internal sources of sulfate that results in either expansive disruption in the paste phase or fills available air voids, reducing freeze-thaw resistance.	1–5 years	Minimize internal sources of slowly soluble sulfates, minimize tricalcium aluminate content in cement, avoid high curing temperatures.
Corrosion of Embedded Steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel.	Chloride ions penetrate concrete, resulting in corrosion of embedded steel, which in turn results in expansion.	3–10 years	Reduce the permeability of the concrete, provide adequate concrete cover, protect steel, or use corrosion inhibitor.

The research approach was to conduct a thorough review of existing literature to establish the state-of-the-practice for the field evaluation, sampling, laboratory analysis, treatment, and prevention of concrete pavement MRD in concrete pavements. From the information gathered, draft guidelines were developed and a field and laboratory study was conducted on six real world pavements suffering unidentified distress(es) in order to test the applicability of the draft guidelines. These guidelines were then modified accordingly.

The three guidelines, which are provided in Volume 2 of this Final Report, provide the primary product of this study. To assist in dissemination of the information contained in the guidelines, a technology transfer package was developed to provide training materials and visual aids needed to conduct a one-day training course.

The literature review suggested what the field evaluation confirmed, that the visual manifestations on the pavement surface of many of these deterioration mechanisms appear similar, especially early in their development. Cracking and staining in the vicinity of joints is an indicator of MRD, but this visual analysis alone does not provide positive identification of what mechanism is at work. For example, it is believed in the past that some cases of external sulfate attack, possibly resulting from deicer impurities, may have been misdiagnosed as aggregate freeze-thaw deterioration (D-cracking).

To address the difficulties of accurate diagnosis, standardized diagnostic methods executed by well-trained personnel are required. This includes training of both the field crews collecting visual information and concrete samples, and the laboratory staff carrying out chemical and petrographic analyses. In some cases, advanced analytical methods based on the use of the scanning electron microscope (SEM) and x-ray diffraction (XRD) may be required to establish what mechanism(s) is at work. This process is illustrated in figure 1. It also must be fully understood that the complexity of the problem might be so great that the best result of a diagnostic investigation is a prioritized list of probable causes.

As mentioned, the guidelines were applied at the six sites with the cooperation and support of the SHAs. Both *Guideline I – Field Distress Survey, Sampling, and Sample Handling Procedures for Distressed Concrete Pavements* and *Guideline II – Laboratory Testing, Data Analysis, and Interpretation Procedures for Distressed Concrete Pavements* were evaluated. The third guideline, *Guideline III – Treatment, Rehabilitation, and Prevention of Materials-Related Distress in Concrete Pavements*, was not applied and is presented as a state-of-the-practice based on a review of the available literature. In

general, the guidelines seemed to direct the necessary work well and provide a systematic method of gathering and recording data.

The first guideline presents a systematic approach for performing a field distress survey, sampling the distressed pavement, and sample handling procedures. In applying this guideline, it was noted that in many instances the construction records for the selected sites were incomplete or limited. In part this may be due to the age of the pavements. However, it may be indicative of a systemic lack of methods and procedures for accurately recording construction data. Even data as fundamental as the job mix formula for the mix design was unavailable in many cases. Information such as climatic conditions during placement is non-existent. Improving data collection, most probably by an automated data collection system during concrete placement, could greatly add to the information available to help diagnose the causes of pavement distress, including MRD. It is suggested in Guideline III that SHAs adopt a more rigorous data collection and storage methodology in line with what is presented in ACI 126.1R, *Guide to a Recommended Format for the Identification of Concrete in a Materials Property Database*.

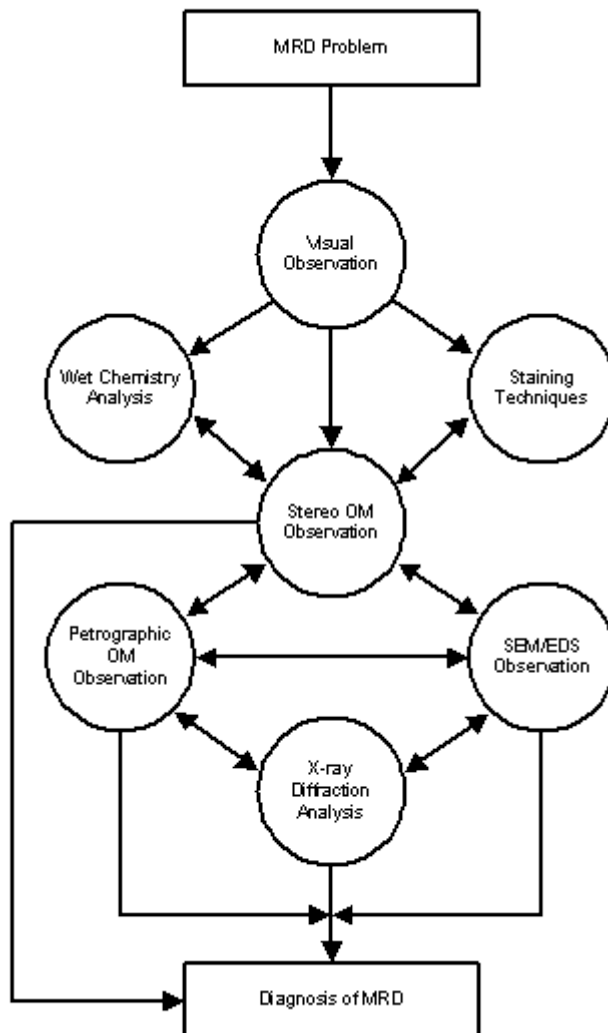


Figure 1. Fundamental process for analyzing a concrete MRD sample.

The application of Guideline I provided a detailed assessment of the current condition of the pavement. This not only provides the current information needed for analysis, but also provides a baseline for monitoring the rate of pavement deterioration when compared to data gathered in the future. This greatly improves the ability of the engineer to maintain the pavement and extend its life, while providing a means to judge the effectiveness of various treatments. This also illustrates one positive aspect of the developed guidelines. They are intended to be applied together, but can easily be applied separately at different times. As an approach, a SHA may use Guideline I as a means of screening pavements and prioritizing maintenance and reconstruction after the specific MRD(s) present has been identified.

The second guideline is the heart of the research effort, as it proposes an approach to laboratory analysis and diagnosis of MRD. The recommended laboratory procedures provide a systematic method for analyzing distressed concrete based on diagnostic flowcharts and tables, which provide a step by step approach to use when trying to determine the exact cause of MRD. Clearly there will be cases where the guidelines fail to isolate the cause to any one MRD mechanism and, in many cases, multiple MRD mechanisms will be identified as possible contributors to the observed distress. However, it is believed that in most cases the data collected using the methods discussed in Guideline II provided a more complete understanding of the distress mechanisms at work. Based upon the results of this evaluation, the majority of cases were resolved. In four of the six case studies used to evaluate the guidelines, definitive and most probable causes of MRD were established. Of the other two, one was identified through execution of the guidelines as not likely being affected by an MRD. It is noted that the laboratory investigation conducted on this site bore out this conclusion, even though early stages of MRD were observed, but were not yet (and may never be) associated with microcracking. In the last site presented, a diagnosis could not be reached using the guidelines, as it became evident early in the evaluation that a different approach would need to be taken to investigate the problem.

The researchers were satisfied with Guideline II in terms of its efficacy and broad applicability. However, in a couple of instances, techniques not proposed in the guideline were employed. Specific examples are the use of epifluorescence microscopy as a means of estimating the effective w/c for the concrete and the use of a flat bed scanner as a low cost imaging tool. Neither of these techniques is precluded by the guidelines and, further, the methods proposed in the guidelines were never intended to be the sole methods of analysis or interpretation. They are simply designed to provide guidance for the common methods.

For engineers working on this project, Guideline II proved to be very useful for helping them understand the process of laboratory analysis. For many engineers, this process is a mystery, and misunderstandings can result if the person interpreting or otherwise using the data does not understand the procedures used. When following the guideline, the choice of tests was understood and the engineers knew that the laboratory personnel progressed through the diagnosis without stopping at the first distress identified.

Unfortunately, for laboratory personnel familiar with the various analytical techniques, the guidelines were reported to be too confining. Laboratory personnel examining concrete are, in general, slow to rush to judgment. The inherent variability in concrete, and the limited sampling possible from most pavements, makes it difficult for an analyst to make yes/no decisions about observations, as is required in the diagnostic flowcharts presented. Laboratory personnel are more comfortable with decisions that are not absolute or are somehow weighted for their significance. As the recipient of the data, the engineer has to understand that absolute decisions are rare and that, in the end, the petrographer or analyst can only provide them with their best judgment. However, the laboratory personnel need to understand the engineers needs. Namely, they need to make yes/no decisions about replacement or rehabilitation and therefore, require the clearest possible diagnosis from the laboratory in order to proceed. Performing the laboratory analysis in accordance with Guideline II helps remove ambiguity and provides a comprehensive look at all possible distresses.

A strong point of Guideline II is that it does not force the diagnosis to resolve at one specific cause. Numerous MRD mechanisms can be active and each should be clearly identified, without bias. The guidelines serve as an interface between engineers and laboratory personnel. Although some MRDs will not be unambiguously diagnosed by using the guidelines, the more common distresses will be identified. Even when absolute diagnosis is not possible, the guidelines help the engineer understand the likely possibilities and the tests available to diagnose the problem further by contracting with outside laboratories.

The third and final guideline was based upon the review of available literature. The results of this review suggest that the various strategies used to treat pavements affected by MRD are not very effective. Most treatments are short-term fixes, such as the application of surface sealers in an attempt to slow the ingress of moisture and deleterious compounds. Some suggested treatments, such as the use of lithium salts in treating alkali silica reactivity, show promise. But in general, long-term treatment of a pavement seriously affected by MRD almost always requires major rehabilitation, either through rubblization and overlaying or complete reconstruction.

Thus, the best method to treat MRD is to prevent it. In new construction, it is recommended that an approach be adopted in which the overall quality of the concrete is emphasized. Strength (especially 7-day or 28-day) is only one measure of quality and it is important that other factors, such as permeability, also be considered. The literature strongly emphasized that the use of short-term strength testing (7-day or 28-day) may be complicit in the increased observation of MRD, and that the emphasis should be shifted to producing dense, impermeable concrete having relatively defect-free insoluble paste microstructural characteristics. This requires the use of durable, non-reactive aggregates arranged to minimize the paste fraction. The paste should have low permeability and solubility. The use of high-quality fly ash or ground granulated blast furnace slag may offer advantages in achieving the desired concrete properties. And care must be exercised during all phases of construction to ensure that the concrete reaches its full potential.

To construct truly durable concrete pavements, it is believed that SHA incentives will need to be modified by changing construction specifications and practices to focus on long-term durability, de-emphasizing rapid construction and short-term strength gain unless project constraints absolutely demand "fast track" construction. It is realized that this will lead to an increase in initial costs, putting concrete pavements at a competitive disadvantage if life cycle costing is not considered. Therefore the revision of SHA policies must not be restricted to the engineering level, but also must include a commitment to accept higher initial costs to achieve high-performance, durable concrete pavements that will provide many years of maintenance-free service. Without this commitment, it is unlikely that proposed changes can be implemented.

In closing, this project has led to the development of three guidelines that should be useful to both SHAs and the private sector to assist in the diagnosis, treatment, and prevention of MRD in concrete pavements. It is acknowledged that the guidelines are not the final authority on this issue, but are simply an attempt to provide standardization and guidance. In future years, it is likely that the body of knowledge in this area of study will continue to grow, and it is anticipated that the guidelines will grow as well, reflecting advancements in laboratory equipment, procedures, and interpretive abilities.

Chapter 1. Introduction

1.1 Project Description

Well-designed and constructed PCC pavements are inherently durable and are expected to be relatively maintenance free during many years of service. Primarily because of these two properties — durability and low maintenance concrete — has been the material of choice for premium pavements for many years.

Unfortunately, a number of pavement distresses can occur as a result of the interaction between the concrete and the environment in which it serves. In contrast to the common perception held by many engineers that concrete is relatively inert, it is in fact a very complex material whose properties can change significantly with time. Some of these changes can be positive, such as long-term strength gain obtained through continued cement hydration. Other changes can be detrimental to the concrete, resulting in the development of premature pavement distress. Paste deterioration resulting from freezing and thawing, aggregate freeze-thaw deterioration (also referred to as D-cracking), and alkali-aggregate reactivity (AAR) are a few examples of detrimental changes that can occur over time. When these changes manifest themselves on the pavement surface, they fall under the general category of MRD.

In general, MRD refers to concrete failures that are a direct result of the properties of the material and its interaction with the environment to which it is exposed. In this sense, these failures are differentiated from others that may be most closely associated with inadequate design for the traffic and environmental loading or the use of improper practices during pavement construction. When considering concrete as a material, it is convenient to think of it as a four-component system consisting of aggregates, cement paste, void space, and pore solution (Leek et al. 1995). And, although traffic and construction may affect the progression of MRD, the initiation mechanisms reside in the properties of the concrete constituents and their interaction with the environment.

MRD in concrete pavement is a concern to all SHAs in the United States. Although the specific type of distresses may vary geographically, it is clear that a better understanding of these types of pavement failures is an important starting point for the production of pavements having longer expected service lives. A principal driving force for understanding the sources of MRD is an increased awareness on the part of SHAs that such failures occur. However, the ability of these agencies to analyze, diagnose, remedy, and prevent these failures has not been fully developed, and this research project focuses on these general areas.

1.2 Objectives

The objectives of this research were to develop guidelines for:

- The systematic field evaluation and representative sampling for laboratory study of distressed areas of concrete pavement.
- The laboratory evaluation of the concrete pavement samples in order to determine the cause(s) of the distress.
- The appropriate treatments of the distresses in existing pavement and prevention of the distresses in new pavements.

This *Volume 1: Final Report* presents the synthesis of background material, describes the development of the guidelines, and briefly introduces the case studies. The other two reports are:

- *Volume 2: Guidelines Description and Use.*
- *Volume 3: Case Studies Using the Guidelines.*

1.3 Scope of Work

The scope of work for this project includes the determination, based on a review of existing literature, of the state-of-the-practice for the field evaluation, sampling, laboratory analysis, treatment, and prevention of concrete pavement MRD. MRD, as identified in the contract, includes, but is not limited to, the following:

- Freeze-thaw deterioration of the paste.
- Alkali–aggregate reactivity.
- D-cracking.
- Sulfate attack.
- Delayed ettringite formation.
- Effect of chlorides on the concrete.

From the information gathered, draft guidelines were developed and a field and laboratory study was conducted on several existing pavements suffering unidentified distress(Es) in order to test the applicability of the draft guidelines. Upon completion of the field and laboratory study, the guidelines were revised based on the findings. The three guidelines provide the primary product of this study, and will be used in the investigation and treatment of pavements affected by MRD, and in providing guidance to prevent MRD in new construction. To assist in dissemination of the information contained in the guidelines, a technology transfer package was developed to provide training materials and visual aids needed to conduct a one-day training course.

The Scope of Work was completed in the following three tasks:

- Task A: Collect Current Information – This task included the formation of a technical advisory panel (TAP) consisting of personnel from SHAs and industry. Relevant available information was collected and synthesized, forming the basis for the development of the draft guidelines. Suitable project test sites were selected to validate the applicability of the developed draft guidelines during the next phase of the study. An interim report was developed and revised reflecting the FHWA's comments, presenting the results of the literature review and the draft guidelines themselves.
- Task B: Evaluation and Validation of the Guidelines – Once written authorization was obtained to proceed, validation of the guidelines commenced. This included intensive field and laboratory evaluation of selected project test sites. The validity and applicability of the draft guidelines was presented to the Federal Highway Administration (FHWA) and TAP, and recommendations were made to improve the guidelines to ensure not only their technical soundness but also their user-friendliness.
- Task C: Preparation of Guidelines, Recommendations, and Final Report – Based on the results of task B and FHWA review, the final version of the guidelines, the project final report, and a technology transfer package were prepared and submitted.

1.4 Report Organization

This *Volume 1, Final Report* provides background information and research results used to develop the guidelines that are contained in Volume 2. Volume 3 presents the case studies that were conducted as part of this study. A technology transfer package was also developed and bound separately.

Volume 1 consists of six chapters. Chapter 1 provides an introduction to the project, including a brief description, the objectives, and the scope. Chapter 2 provides background information on MRD types commonly affecting concrete pavements. Chapter 3 discusses the need and development of the guidelines, including the background information used in their development. Chapter 4 presents the evaluation of the field and laboratory guidelines, and chapter 5 summarizes the report and presents the conclusions drawn from the evaluations conducted. Chapter 6 lists the cited references. Appendix A provides detailed information regarding the distress mechanisms that is not appropriate for inclusion in the body of the report, but will be of value to the interested reader.

Chapter 2. MRD In Concrete Pavements

2.1 Background

Whether a concrete pavement will be affected by MRD is a function of many factors, including the constituent materials used (aggregate, cement, additives, and so on), the location (maritime or inland), the climatic conditions (temperature, moisture), and the presence of external agents (such as roadway deicing chemicals or sulfates in groundwater). In general, the development of MRD can be attributed to either physical or chemical mechanisms, although the two often act together to initiate and propagate distress. The large number of factors and their complex interactions make it difficult in many cases to determine a single "cause" of the observed pavement distress, as more than one mechanism may be at work.

MRD is generally manifest on the pavement surface as fine cracking or as material degradation such as scaling or spalling. The distress may be isolated in the vicinity of joints or cracks or distributed over the pavement surface. Discoloration of the concrete, sometimes referred to as staining, is also a common feature, as is the observation of exudate in cracks. In some instances, concrete expansion may occur, resulting in blowups at joints and displacement of fixed structures. Depending upon the type of distress and the environment to which the pavement is exposed, these distresses may occur as soon as a few years following construction. In other cases, noticeable pavement deterioration will not be observed until much later in the pavement's service life.

MRD has received greater attention in recent years, in part because of an increased recognition of the importance of material durability in the performance of concrete pavements. Whereas previously it was commonly assumed that sufficient concrete strength ensured durability, it is now recognized that concrete mixtures must be designed for both strength and durability. In addition, it is common today to extend the anticipated concrete pavement design life well beyond the 20 years that was common in the past. This heightens the need for concrete durability to ensure satisfactory pavement performance for the desired 40 to 60 years of service.

Interest in concrete durability has existed almost as long as concrete has been used. Innumerable conferences and thousands of publications have focused on various aspects of concrete durability. Contrary to popular belief, durability is not an intrinsic property of the concrete. One of the most authoritative compilations of this vast amount of information is the *Guide to Durable Concrete* [American Concrete Institute (ACI) 1992a], which cites over 250 references. In this guide, the "durability of hydraulic-cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration." In a recent Transportation Research Circular, entitled the *Durability of Concrete* [Transportation Research Board (TRB) 1999], durable concrete is "concrete that in the particular environment of service resists the forces in that environment that tend to cause it to disintegrate without requiring excessive effort for maintenance during its service life." The second definition is more descriptive, as durability is linked not only to the properties of the concrete, but also to the service conditions to which it is subjected during its service life. This concept is captured in the statement that "a given concrete with a given set of properties will endure without noticeable change for centuries or even millennia in one environment and be reduced to fragments in a few years or even a few months in another" (TRB 1999). This linkage between concrete properties and the environment is key to defining what constitutes durability and thus the occurrence of MRD.

In this project, only MRD that commonly affects concrete pavements has been considered. In contrast to the range of deterioration mechanisms considered in the *Guide to Durable Concrete* (ACI 1992a), some distress mechanisms were not included in this study in recognition that they will only rarely, if ever, affect concrete pavements (acid attack and seawater exposure, for example). Others were not included because the primary cause of deterioration is not directly a result of the interaction between the concrete and environment factors (e.g., moisture, temperature, or external agents). Abrasion is one such distress included in the *Guide to Durable Concrete* that is not included in this study, since its incidence is primarily linked to the use of tire chains and studded snow tires, which "cause considerable wear to concrete

surfaces, even where the concrete is of good quality" (ACI 1992a). Abrasion is thus considered to be primarily related to the loading condition, and is not directly attributable to the interaction between the material and the environment.

2.2 Types of MRD

As discussed, distresses that are directly related to the interaction between the concrete materials used to construct a pavement and the environment are referred to as MRD. The development of MRD can be attributed to either physical or chemical mechanisms, although the two types of mechanisms often act together to bring about the development of distress. Furthermore, MRD due to multiple causes may develop simultaneously, thereby complicating the determination of the exact cause(s) of material failure.

The following MRD types are considered in this study:

- Freeze-thaw deterioration of hardened cement paste.
- Deicer scaling/deterioration.
- Freeze-thaw deterioration of aggregate.
- Alkali-silica reactivity (ASR).
- Alkali-carbonate reactivity (ACR).
- External sulfate attack.
- Internal sulfate attack.
- Corrosion of embedded steel.

The first three MRD types listed have been categorized as being caused by physical mechanisms and the remaining five by chemical mechanisms. Although this list presents eight distinct MRD types, the complexity of the physical and chemical interactions involved in the development and manifestations of some types of MRD, as well as gaps in the current knowledge base, often prevent absolute identification of the primary mechanism of distress. Also, the fact that more than one distress mechanism may be at work at the same time further complicates the identification process. And finally, it is noted that there are less common occurrences of other types of MRD that are not listed, but may also play a role in concrete pavement deterioration.

Table 1-1 summarizes these distresses, along with general information on their causes, appearance, and prevention; a description of these distresses is provided below. A more detailed description of the distress mechanisms is presented in Appendix A of Volume 1 of the Final Report.

Deterioration Due to Physical Mechanisms

Freeze-Thaw Deterioration of Hardened Cement Paste

Freeze-thaw deterioration of hardened cement paste is caused by the deterioration of saturated cement paste under repeated freeze-thaw cycles. Currently, there is no consensus on the exact mechanisms responsible for internal damage resulting from freeze-thaw action. The most widely accepted theories consider the development of internal tensile stress as a result of either hydraulic pressures, osmotic pressures, or a combination of the two during freezing.

In early efforts to understand freeze-thaw action in concrete, Powers (1945) attributed freeze-thaw damage to excessive hydraulic pressures produced from the expansion of ice. It was proposed that, as ice forms in the pore system, the resulting 9 percent volume expansion causes the surrounding unfrozen water to be expelled from the freezing sites. Depending on the nature of the pore system, excessive internal stresses can develop from hydraulic pressures that are incurred due to resistance to this flow.

More recent theories (Powers 1975) consider osmotic potential to be the primary cause of excess internal stress. As pure water in the larger pores freeze, the liquid remaining in the pores becomes a more concentrated ionic solution. To maintain thermodynamic equilibrium, the less concentrated solution in the surrounding paste is drawn to the freezing sites. If adequate air void space is available, all of the freezable water will eventually diffuse to the freezing sites inside the air voids, reaching a state of equilibrium. If the air void space is inadequate, equilibrium cannot be reached and osmotic pressures sufficient to fracture the paste can result.

Deterioration of the cement paste due to freeze-thaw damage manifests itself in the form of scaling, map cracking, or severe cracking, spalling, and deterioration, commonly initiating at joints and free edges where moisture is more readily available. The addition of an air-entraining agent (an admixture that stabilizes a system of microscopic bubbles in the concrete) is an effective means of preventing this deterioration. This is commonly a top down distress with fractures running nominally parallel with the pavement surface, decreasing in number with depth.

Table 1-1. Summary of key MRDs.

Type of MRD	Surface Distress Manifestations and Locations	Causes/ Mechanisms	Time of Appearance	Prevention or Reduction
<i>MRD Due to Physical Mechanisms</i>				
Freeze-Thaw Deterioration of Hardened Cement Paste	Scaling, spalling or map-cracking, generally initiating near joints or cracks; possible internal disruption of concrete matrix.	Deterioration of saturated cement paste due to repeated freeze-thaw cycles.	1 to 5 years	Addition of air-entraining agent to establish protective air-void system.
Deicer Scaling/Deterioration	Scaling or crazing of the slab surface with possible alteration of the concrete pore system and/or the hydrated cement paste leading to staining at joints/cracks.	Deicing chemicals can amplify freeze-thaw deterioration and may interact chemically with cement hydration products.	1 to 5 years	Provide minimum cement content of 335 kg/m ³ , limit water–cement ratio to no more than 0.45, and provide a minimum 30-day “drying” period after curing before allowing the use of deicers.
Freeze-Thaw Deterioration of Aggregate	Cracking parallel to joints and cracks and later spalling; may be accompanied by surface staining.	Freezing and thawing of susceptible coarse aggregates results in fracturing and/or excessive dilation of aggregate.	10 to 15 years	Use of non-susceptible aggregates or reduction in maximum coarse aggregate size.
<i>MRD Due to Chemical Mechanisms</i>				
Alkali–Silica Reactivity (ASR)	Map cracking over entire slab area, usually with exudate, and accompanying expansion-related	Reaction between alkalis in the pore solution and reactive silica in aggregate resulting in the formation of an	5 to 15 years	Use of nonsusceptible aggregates, addition of pozzolans to mix, limiting total alkalis in concrete, minimizing exposure to moisture,

	distresses (joint closure, spalling, blowups).	expansive gel and the degradation of the aggregate particle.		addition of lithium compounds.
Alkali–Carbonate Reactivity (ACR)	Map cracking over entire slab area and accompanying pressure-related distresses (spalling, blowups).	Expansive reaction between alkalis in pore solution and certain carbonate/dolomitic aggregates which commonly involves dedolomitization and brucite formation.	5 to 15 years	Avoid susceptible aggregates, significantly limit total alkalis in concrete, blend susceptible aggregate with quality aggregate or reduce size of reactive aggregate.
External Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area, ultimately resulting in joint or surface deterioration.	Expansive formation of ettringite that occurs when external sources of sulfate (e.g., groundwater, deicing chemicals) react with the calcium sulfoaluminates.	3 to 10 years	Use w/c below 0.45, minimize tricalcium aluminate content in cement, use blended cements, use pozzolans.
Internal Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Formation of ettringite from internal sources of sulfate that results in either expansive disruption in the paste phase or fills available air-voids, reducing freeze-thaw resistance.	1 to 5 years	Minimize internal sources of slowly soluble sulfates, minimize tricalcium aluminate content in cement, avoid high curing temperatures.
Corrosion of Embedded Steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel with rust present.	Chloride ions penetrate concrete, resulting in corrosion of embedded steel, which in turn results in expansion.	3 to 10 years	Reduce the permeability of the concrete, provide adequate concrete cover, protect steel, or use corrosion inhibitor.

Deicer Scaling/Deterioration

Deicer scaling/deterioration is typically observed as scaling or crazing of the slab surface due to the repeated application of deicing chemicals. Although the exact causes of deicer scaling are not known, it is commonly believed to be primarily a physical attack. The primary mechanisms considered in the physical deterioration models are High thermal strains produced when a deicer melts ice and/or high osmotic pressures induced when relatively pure surface water attempts to equalize highly concentrated salt solutions present in the concrete (Mindess and Young 1981; Pigeon and Plateau 1995). It has also been speculated that pressure exerted by salt crystallization in voids is a contributing factor (Hansen 1963). Recent studies suggest that chemical alteration of the cement paste may also be occurring, resulting in dissolution of calcium hydroxide, coarsening of the concrete pore system and, potentially, the formation of deleteriously expansive compounds. This occurrence is commonly observed as staining and deterioration in the vicinity of joints.

Deicer scaling/deterioration is more likely to occur in concrete that has been over-vibrated or improperly finished, actions that create a weak layer of paste or mortar either at or just below the surface (Mindess and Young 1981). This weakened layer is more susceptible to hydraulic pressures. Even adequately air-

entrained concrete can be susceptible to the development of salt scaling. Recommendations for the prevention of salt scaling include providing a minimum cement content of 335 kg/m³, limiting the water–cement ratio (w/c) to a maximum of 0.45, providing adequate curing, and providing an absolute minimum of 30 days of environmental exposure before allowing the application of deicing chemicals [American Concrete Pavement Association (ACPA) 1992].

Freeze-Thaw Deterioration of Aggregate

Freeze-thaw deterioration of aggregate is a distress associated with the freezing and thawing of susceptible coarse aggregate particles in the concrete. This phenomenon is commonly referred to as D-cracking in pavements [Strategic Highway Research Program (SHRP) 1993], and aggregates are typically identified as being D-cracking susceptible. Such aggregates either fracture and/or dilate as they freeze, resulting in cracking of the surrounding mortar. It has also been hypothesized in some cases that the expulsion of water during freezing contributes to dissolution of soluble paste components, such as calcium hydroxide, in the interfacial zone. Key aggregate properties related to susceptibility are composition, pore structure, sorption, and size (Schwartz 1987). Most susceptible aggregates are of sedimentary origin and are most commonly composed of limestone, dolomite, or chert (Stark 1976).

Freeze-thaw deterioration of aggregate is initially visible as a series of fine cracks generally running parallel to joints, cracks, or free edges in the slab. Deterioration commonly starts near the bottom of the concrete slab where excess moisture accumulates. As the number of freeze-thaw cycles increases, spalling and deterioration of the cracks will occur. A dark staining due to calcium hydroxide or calcium carbonate residue generally precedes and accompanies the cracking, often in an hourglass shape on the pavement surface at affected joints and cracks.

Air entrainment of the cement paste does not prevent the development of D-cracking. The best means of preventing this distress is by prohibiting the use of susceptible aggregate, although reducing the maximum size of the susceptible coarse aggregate has been shown to be effective in reducing freeze-thaw deterioration of aggregate in many instances.

Deterioration Due to Chemical Mechanisms

Alkali–Silica Reactivity (ASR)

Alkali–silica reactivity (ASR) is most commonly associated with undesirable chemical reactions between alkalis in the cement paste (commonly reported as percent Na₂O plus 0.658 x percent K₂O) and the reactive siliceous components of susceptible aggregates. It is the concentration of the hydroxyl ion in the concrete pore solution that is of interest, which is related to the alkali-content (ACI 1998). The product of the reaction is a gel that significantly expands in the presence of moisture, destroying the integrity of the weakened aggregate particle and the surrounding cement paste. An irregular, map-like cracking ultimately develops, most often over the entire slab area (with cracks generally less than 50 mm deep). ASR can also lead to internal horizontal cracks at greater depths within the slab. Upon continued expansion, joint spalling, blowups, shoving of fixed structures, and other pressure-related distresses in the pavement can occur. A handbook depicting ASR distress in pavements and highway structures is available to aid in its identification (Stark 1991).

The chemical reactions occurring during the development of ASR are very complex, but three basic conditions are needed in order for ASR to occur (Farny and Kosmatka 1997):

- Reactive forms of silica in the aggregate.
- High-alkali concrete pore solution.
- Sufficient moisture.

The concrete pore solutions are primarily alkali hydroxide solutions of high concentration, which react readily with reactive forms of silica (Stark et al. 1993). As the aggregate reactivity increases, gel reaction products can be formed with lesser concentrations of alkali (Farny and Kosmatka 1997). As the alkalinity (and the pH) of the pore solution increases, the potential for alkali-silica reaction increases as even more stable forms of silica become susceptible to attack. The presence of moisture allows migration of alkali ions to reaction sites and the resulting gel absorbs moisture, leading to expansion (Stark et al. 1993). Relative humidity levels above 80 percent indicate that moisture is sufficiently available for absorption by ASR gel, and recent research has shown that pavement concrete in all climates likely will be sufficiently damp to continuously support expansive ASR (Stark et al. 1993).

Other factors influencing the development of ASR include the total alkali content of the concrete, the presence of external sources of alkalis (e.g., chemical deicers), repeated cycles of wetting and drying, and temperature (Farny and Kosmatka 1997, ACI 1998). A more detailed description of the ASR reaction process is provided by Helmuth (1993).

The gel that is produced through the reaction appears as a glassy-clear or white powdery deposit within reacted aggregate particles, although it is not always visible to the naked eye (Stark et al. 1993). The presence of the reaction product (which is an alkali–calcium–silica–hydrate gel) does not always coincide with distress, and thus gel presence by itself does not necessarily indicate destructive ASR (Farny and Kosmatka 1997).

Common aggregate types containing reactive silica components include opaline or chalcedonic cherts, siliceous limestones, rhyolites and rhyolitic tuffs, dacites and dacite tuffs, andesites and andesite tuffs, and phyllites (Dolar-Mantuani 1982; Neville 1996). An excellent summary of alkali-silica reactive aggregates is presented in the ACI State-of-the-Art Report on Alkali-Aggregate Reactivity (ACI 1998). The rate of the reaction will vary considerably among aggregates, with some undergoing a complete reaction within a matter of weeks and others requiring many years to produce noticeable effects (Helmuth 1993).

A variety of approaches have been tried to prevent or minimize the development of ASR, with mixed success. In new concrete designs, the use of low-alkali portland cement (with an alkali content less than 0.60 percent Na_2O equivalent) has been used successfully on slightly to moderately reactive aggregates (Farny and Kosmatka 1997). Some international agencies limit the alkali content of the concrete, accounting for the cement factor and other internal sources of alkalis (ACI 1998). The addition of fly ash has also been shown to control ASR, although this is strongly dependent upon the type of fly ash, its alkali content, chemical composition, and dosage rate (Farny and Kosmatka 1997). Other types of finely divided materials can also be effective including silica fume, ground granulated blast furnace slag, and natural pozzolans (ACI 1998). Finally, the addition of ASR-inhibiting compounds (e.g., lithium hydroxide) has been shown to be effective on highly reactive aggregates (Stark et al. 1993).

For in-service concrete displaying ASR, no definitive method has been identified that is completely successful in stopping the distress mechanisms. The application of silane sealers, methacrylate, and lithium hydroxide have all been tried, with monitoring of the effectiveness of these procedures ongoing (Stark et al. 1993).

Alkali–Carbonate Reactivity (ACR)

Alkali–carbonate reactivity (ACR) is another distress caused by an undesirable chemical reaction between the pore solution and aggregate; in this case, the reaction is between the alkalis in the pore solution and certain dolomitic carbonate aggregates containing a characteristic reactive texture of dolomite rhombs in a clayey fine-grained matrix. Although the mechanisms for ACR are not as well understood as those for ASR, it has been established that dedolomization occurs, that is, the decomposition of dolomite into calcium carbonate and magnesium hydroxide, which is accompanied by expansion. This expansion may be due to a combination of migration of alkali ions and water molecules

into the restricted space of the fine-grained matrix surrounding the dolomite crystal, migration of these materials into the crystal, and the growth and arrangement of the dedolomitization products, especially brucite (Farny and Kosmatka 1997, ACI 1998). Other factors influencing the development of ACR include maximum size of the reactive aggregate (rate/degree of expansion decreases with decreasing aggregate size) and pore solution alkalinity (increasing pH levels increase the potential for alkali-carbonate reactions) (Farny and Kosmatka 1997; ACI 1998).

Similar to the expansive pressures that are developed in ASR distress, the expansive pressures developed by ACR also result in map-like cracking on the pavement surface and accompanying expansion-related distresses (spalling, blowups). Avoiding the use of susceptible aggregates is the one sure way of avoiding ACR, but either diluting susceptible aggregates with non-susceptible aggregates or reducing the maximum size of susceptible aggregates can minimize the deleterious effect of the reaction (ACI 1998). Limiting the alkali content in the cement is another method employed to prevent ACR, but the alkali content must be lower than that typically used to prevent ASR (ACI 1998). However, unlike ASR, pozzolans are not effective in controlling alkali-carbonate reaction (Farny and Kosmatka 1997; ACI 1998).

External Sulfate Attack (ESA)

External sulfate attack (ESA) results from chemical reactions that occur when concrete is exposed to penetrating external sulfate ions (present in groundwater, soil, deicing chemicals, etc.). Although the mechanism of sulfate attack is complex, it is primarily thought to be caused by two chemical reactions: 1) the formation of gypsum through the combination of sulfate and calcium ions, and/or 2) the formation of ettringite through the combination of sulfate ions and hydrated calcium aluminate (ACI 1992a, DePuy 1994). In either case, the formation of the reaction product leads to an increase in solid volume. In the former case, expansion due to gypsum formation may not be destructive, but gypsum has little cementing properties and thus the concrete loses integrity (DePuy 1994). On the other hand, the expansive pressures exerted by ettringite formation can be very destructive.

In concrete pavements, deterioration due to external sulfate attack typically first appears as cracking near joints and slab edges, generally within a few years of construction. Fine longitudinal cracking may also occur parallel to longitudinal joints. Steps taken to prevent the development of distress due to external sulfate attack include minimizing the tricalcium aluminate content in the cement or reducing the quantity of calcium hydroxide in the hydrated cement paste through the use of pozzolanic materials. It is also recommended that a *w/c* ratio less than 0.45 will help mitigate external sulfate attack (ACI 1992a).

Internal Sulfate Attack (ISA)

Internal sulfate attack (ISA) is a potential pavement distress similar in many ways to external sulfate attack, except that the source of the sulfate ions is internal. Internal sources of sulfate include slowly soluble sulfate contained in clinker, aggregate, and admixtures (such as fly ash) or as a result of decomposition of primary ettringite during early hydration, primarily associated with high curing temperatures. This particular distress has elicited considerable debate among concrete material experts regarding the specific mechanisms of distress and the precise role of ettringite in its development. Sometimes called delayed ettringite formation (DEF) distress or secondary ettringite formation (SEF) distress, it is referred to here as internal sulfate attack to distinguish the source of sulfate ions. For consistency and clarification, the following definitions are offered regarding the various forms of ettringite (based on Erlin 1996a):

Ettringite—a high-sulfate calcium sulfoaluminate mineral ($3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 32\text{H}_2\text{O}$).

Primary Ettringite—ettringite formed by reaction of sulfate and aluminate ions during early hydration of hydraulic cement either as a normal process for portland cement or as the expansive process for expansive cement.

Secondary Ettringite—ettringite commonly formed in available void space by precipitation from solution of either primary or delayed ettringite.

Delayed Ettringite—ettringite formed by reaction of sulfate and aluminum ions in concrete, mortar, or grout that has hardened and developed its intended strength; the source of the sulfate ions is from within the concrete.

It is believed that both SEF and DEF are forms of internal sulfate attack, but result for different reasons. SEF is commonly a product of concrete degradation, characterized by the dissolution and subsequent precipitation of ettringite into available void space and in pre-existing microcracks. SEF can occur in concrete that is sufficiently permeable and saturated, allowing the dissolution and precipitation process to occur. Although most experts agree that secondary ettringite formation will not generate sufficient expansive pressures to fracture healthy cement paste or mortar, its presence in the air void structure may limit the ability of the paste to resist freeze-thaw deterioration (Ouyang and Lane 1999). Thus, concrete that appears to be suffering paste freeze-thaw deterioration may have originally had an adequate air void system that has been compromised by SEF. Yet there remains considerable debate as to whether the SEF is the cause of distress or only present as a result of another deterioration mechanism that disrupted the paste sufficiently to encourage dissolution and precipitation of the ettringite (Detwiler and Powers-Couche 1999).

DEF, on the other hand, can lead to destructive expansion within the paste, resulting in microcracking and separation of the paste from aggregate particles. DEF is most often associated with steam curing. At elevated temperatures (current research suggests a minimum temperature of 65 °C to 80 °C, with many citing 70 °C [Scrivener 1996; Thaulow et al. 1996a; Klemm and Miller 1999]), primary ettringite will not form properly. After the concrete has cured and temperatures are reduced to ambient conditions, sulfates and aluminate phases in the paste may then react to form expansive ettringite, disrupting the concrete matrix. Because this phenomenon is most closely associated with steam curing, it is still speculative whether cast-in-place pavements can experience the temperatures necessary to produce DEF. It has also been suggested that internal sulfate attack might occur due to internal sources of sulfate that become available after the paste has hardened. Possible sources include either slowly soluble sulfates or sulfur compounds in the clinker or fly ash that only become available during continued long-term hydration. Another internal source of sulfates might be from aggregates (Johansen and Thaulow 1999). In these cases, the internal sulfate attack is not from DEF, but instead from excess sulfates in the concrete mixture, which result in paste expansion along similar lines as external sulfate attack.

The manifestation of internal sulfate attack in many concrete structures is characterized by a series of closely spaced, tight map cracks with wide cracks appearing at regular intervals. Microscopically, paste expansion due to DEF can be identified by gaps in the paste around aggregates with widths proportional to the particle size (Johansen and Thaulow 1999).

Only recently have researchers started investigating internal sulfate attack as a potential pavement distress mechanism, and possible means of prevention are still being explored. Recommendations from a recently completed study indicate that limiting the sulfate content of cement and fly ash may assist in preventing ISA, but that it is impossible to recommend a specification at this time (Gress 1997). Other studies suggest that limiting sulfate content is only an issue if high curing temperatures are to be expected (Scrivener 1999). The extent of the problem in pavements appears to be limited at this point, but additional research is necessary to determine both the extent of internal sulfate attack and feasible strategies to prevent its occurrence in new pavements.

Corrosion of Embedded Steel

Corrosion of embedded steel appears as rust colored staining, spalling, cracking, and associated deterioration of the concrete above or surrounding the steel affected by active corrosion. Steel corrosion is accelerated in the presence of chloride ions (which can come from a chloride-based accelerator added

to the mix, deicing salts, or seawater) that break down the passivity film that protects embedded steel from corrosion.

Corrosion of embedded steel is controlled by prohibiting chloride additions to fresh concrete, decreasing permeability of the concrete, providing adequate concrete cover over the steel, coating steel with a protective layer, application of a protective coating on the concrete surface to prevent penetration of chlorides and moisture, and suppression of the electrochemical process at the steel surface.

2.3 Advisory Guidelines

Due to the complexity of the distress mechanisms at work, the specialized laboratory methods employed, and the lack of standardization in the evaluation process, it became apparent that guidance was needed to assist SHAs in the evaluation of MRD affected concrete pavements. As part of this project, the following three guidelines were developed to provide pavement engineers and field and laboratory personnel with a systematic procedure for the identification, evaluation, treatment, and prevention of MRD in PCC pavements:

- Guideline I — Field Distress Survey, Sampling, and Sample Handling Procedures for Distressed Concrete Pavements.
- Guideline II — Laboratory Testing, Data Analysis, and Interpretation Procedures for Distressed Concrete Pavements.
- Guideline III — Treatment, Rehabilitation, and Prevention of Materials-Related Distress in Concrete Pavements.

These guidelines have been prepared with the practitioner in mind, and it is expected that the application of a standardized, systematic approach to the overall data collection, analysis, and interpretation process will provide an accurate assessment of the type of distress(es) present and their cause(s). Furthermore, information collected in accordance with the guidelines will prove useful as additional research and investigations are conducted on MRD affected pavements.

Need for Advisory Guidelines

The guidelines have been prepared to assist pavement engineers and field and laboratory personnel in the identification, evaluation, treatment, and prevention of MRD in concrete pavements. Investigations of concrete pavement deterioration have been conducted for years by numerous agencies, laboratories, and institutions, each with their own set of guidelines and testing procedures. Some SHAs have studied MRD in great detail, establishing unique testing protocols and material specifications. Other agencies, however, are not familiar with these procedures. The American Society for Testing and Materials *Standard Practice for Petrographic Examinations of Hardened Concrete* (ASTM C 856) provides an excellent outline of procedures for petrographic evaluation of concrete, but is focused strictly on petrographic means and is not specifically oriented toward the evaluation of MRD affected concrete pavements. In summary, there is a lack of information on the subject, and where information exists, there is little standardization regarding nomenclature, test methods, and identification procedures. The lack of standardization in test methods and data collection procedures makes it difficult to reach consensus on diagnoses and to analyze the performance of concrete pavements on a national basis.

Purpose and Objectives of Guidelines

The overall purpose of the guidelines is to provide a standardized approach for field evaluation, sampling and handling, and laboratory procedures for use in identifying MRD in pavements. Furthermore, these guidelines will provide useful information on effective measures for the economical preservation of PCC pavements identified as suffering MRD and guidance for the construction of long-lasting PCC pavements. It is expected that these guidelines will assist in the free flow of information between various

agencies, laboratories, and institutions so that all benefit from the findings of each. Figure 1-1 illustrates the flow of the three guidelines involved in the identification, treatment, and prevention of MRD.

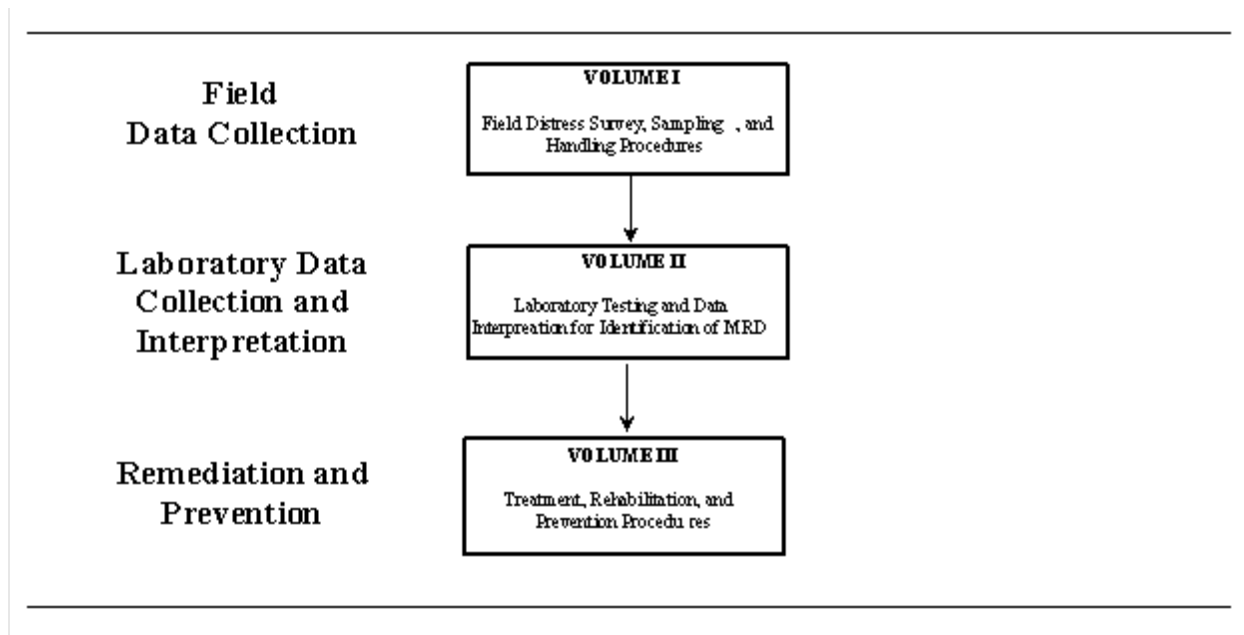


Figure 1-1. Flow of guidelines for identification, treatment, and prevention of MRD.

Chapter 3. Development Of Advisory Guidelines

This chapter briefly summarizes background information used to develop the guidelines. The chapter is divided into three sections corresponding to each of the three guidelines. The complete guidelines are presented in Volume 2 of this Final Report.

3.1 Background for Field Distress survey, Sampling, and Sample Handling Procedures

The identification of MRD generally begins with a field distress survey. During the field survey, the manifestations of observed distress are noted (such as the prevailing crack pattern, the location and extent of the cracking pattern within pavement slabs, the presence of any staining, and the deposition of any exudate). This information can be used to distinguish MRD from other PCC pavement distresses caused by different mechanisms and to suggest possible MRD types. However, the similar distress manifestations of many of the MRD types make it extremely difficult to accurately and positively identify the specific MRD type and its associated mechanisms based on visible distress manifestations alone. Furthermore, a particular pavement may be affected by several MRD mechanisms, thereby further complicating field identification of the MRD. Thus, while field surveys can provide indications of the type(s) of MRD and suggest possible mechanisms, the absolute identification of the MRD can only be accomplished through a detailed laboratory analysis of samples obtained from the pavement.

For the most part, very little information is available regarding the field survey of MRD-affected pavements and the associated field identification of MRD types. This section reviews several common distress survey methodologies to examine the applicability of their procedures for surveying MRD affected pavement while noting how each method differentiates among the various MRD types. Based on that information, the framework adopted in the first guideline for conducting a field survey of PCC pavements exhibiting MRD is described. A summary of additional field testing methods that are potentially available to assist in the field identification of MRD is provided. A discussion of field sampling and handling procedures is then provided, followed by the approach adopted for use in this study.

Review of Selected Distress Survey Methodologies

Over the years, several attempts have been made at developing pavement condition survey procedures and distress identification manuals [Highway Research Board (HRB) 1957; Hveem 1958; HRB 1970; Smith et al. 1979]. However, the increased interest in pavement management activities in the late 1970's and the launching of the SHRP initiative in 1987 established the need for more standardized procedures and distress definitions. For example, many highway agencies have developed procedures for their own conditions and applications, and most of these are intended for network-level condition monitoring [e.g., Washington Department of Transportation (WSDOT) 1983; Pennsylvania DOT (PennDOT) 1996, Florida DOT (FDOT) 1996]. As such, there is little (if any) information on identifying specific manifestations consistent with MRD, although many highway agency distress manuals do include D-cracking and map cracking distress types that are consistent with aggregate freeze-thaw deterioration and ASR, respectively.

Of the various distress survey and identification procedures that have been developed, two stand out as currently being the most widely used and accepted: the SHRP Long-Term Pavement Performance (LTPP) distress identification manual (SHRP 1993) and the Corps of Engineers Pavement Condition Index (PCI) survey procedure (Shahin and Walther 1990). Each of these was developed for a specific purpose, with the SHRP LTPP manual developed for use in the LTPP program and the PCI procedure for network-level pavement management systems (PMS) (although it can also be used for project-level evaluations). The field survey methodology and the way the respective procedures account for MRD are described in the following sections, along with a brief description of an ACI publication on making a condition survey of concrete in service (ACI 1992b).

Description and Purpose

The SHRP LTPP distress identification manual was developed to provide a consistent basis for collecting distress data for the LTPP program (SHRP 1993). Because the pavement sites in the LTPP program are located throughout the country and distress will often be collected by different parties, there was an acute need for a uniform basis in distress data collection to ensure the integrity and the compatibility of the data.

Since it was developed for the LTPP program, the manual is oriented toward research applications. That is, it requires that distress data be collected in considerable detail and at high levels of precision in order for the data to be used in characterizing pavement performance and behavior. However, even though it is a research-oriented manual, it does little to distinguish cracking patterns or physical manifestations common to the various types of MRD.

Survey Procedure

The SHRP LTPP distress identification manual includes an appendix describing the survey procedures, with detailed instructions on how the survey should be conducted. Blank survey forms and standard symbols for noting the various types of distress are included as part of the appendix. However, since the focus of the procedure is on distress surveys of previously identified and delimited test sites, no guidance or information is provided on sampling or sectioning; a previously selected 150-m-long section serves as the basic sample unit length for the procedure.

The condition survey is conducted with the survey team walking over the length of the sample unit and recording the observed distresses on the field distress form. This is accomplished by using the distress symbols and numbers shown in figures 1-2 and 1-3 to indicate the occurrence of the various PCC pavement distress types. The type, amount, and severity of the distresses are noted on the field survey sheets and then later summarized on another data sheet. An example of a completed form is shown in figure 1-4, which represents 30.5 m of the sample unit; additional sheets are needed for the documentation of the entire length of the sample unit. During the field survey, only distresses in the outer traffic lane are noted.

Distress Type	Symbol	Distress Type	Symbol
1. Corner Breaks (Number) L, M, H*		8a. Map Cracking 8b. Scaling (Square Meters) No Severity Levels	
3. Longitudinal Cracking (Meters) L, M, H* S - Sealed		9. Polished Aggregate (Square Meters) No Severity Levels	
4. Transverse Cracking (No. of Cracks and Length) (Meters) L, M, H*		10. Popouts (Number) No Severity Levels	
5a. Joint Seal Damage of Transverse Joints (Number) L, M, H*		11. Blowups (Number) No Severity Levels	
5b. Joint Seal Damage of Longitudinal Joints (Number) No Severity Levels		12. Faulting of Transverse Joints and Cracks (Millimeters)	
6. Spalling of Longitudinal Joints (Meters) L, M, H*		15. Patch/Patch Deterioration (Square Meters and Number) L, M, H* F-Flexible, R-Rigid	
7. Spalling of Transverse Joints (No. of Joints and Length) (Meters) L, M, H*		16. Water Bleeding and Pumping (Number of Occurrences and Length of Affected Pavement) (Meters) No Severity Levels	

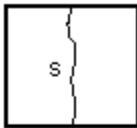
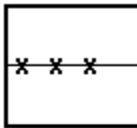
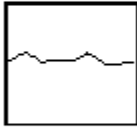
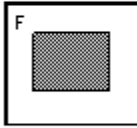
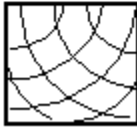
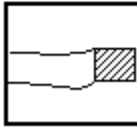
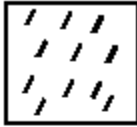
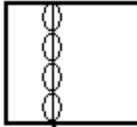
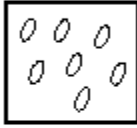
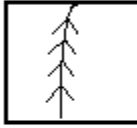
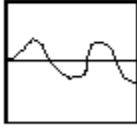
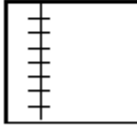
* Low, Moderate, and High Severity Levels

Figure 1-2. SHRP LTPP distress symbols for jointed concrete pavements (JCP) (SHRP 1993).

Table. Description of symbols for figure 1-2.

Distress Number	Distress Name	Symbol Description	Severity Indication	Other Indications
1	Corner breaks	Arc drawn in corner	L, M, H	Number each of the corner breaks
3	Longitudinal cracking	Vertical line at crack site	L, M, H	Length of crack in meters; write the letter S on the map at the distress site if the crack is sealed
4	Transverse cracking	Horizontal line at crack site	L, M, H	Number of cracks and length in meters
5a	Joint seal damage of transverse joints	Shaded box around the horizontal joint where damage is occurring	L, M, H	Number of areas of damage
5b	Joint seal damage of the longitudinal joints	Shaded box around the vertical joint where damage is occurring	No severity levels	Number of areas of damage
6	Spalling of longitudinal joints	Series of small, vertical circles around the joint	L, M, H	Length in meters
7	Spalling of transverse joints	Series of small, horizontal circles around the joint	L, M, H	Number of joints and length in meters
8a	Map cracking	Rounded plaid/fishnet pattern	No severity levels	Square meters of distress
8b	Scaling	Rounded plaid/fishnet pattern	No severity levels	Square meters of distress
9	Polished aggregate	Small diagonal lines	No severity levels	Square meters of distress
10	Popouts	Small ovals drawn at an angle	No severity levels	Number
11	Blowups	Curved line over joints	No severity levels	Number

12	Faulting of transverse joints and cracks	Straight line	No severity levels	Length of faulting in millimeters
15	Patch/patch deterioration	Shaded square over patched area	L, M, H	Square meters and number of patch; indicate on map an F if flexible or an R if rigid
16	Water bleeding and pumping	Arrowhead pattern drawn on the crack	No severity levels	Number of occurrences; length of affected pavement in meters

Distress Type	Symbol	Distress Type	Symbol
2. Longitudinal Cracking (Meters) L, M, H* S - Sealed		8. Transverse Construction Joint Deterioration (Number) L, M, H*	
3. Transverse Cracking (No. of Cracks and Length) (Meters) L, M, H*		11. Patch/Patch Deterioration (Square Meters and Number) L, M, H* F - Flexible, R - Rigid	
4a. Map Cracking 4b. Scaling (Square Meters) No Severity Levels		12. Punchouts (Number) L, M, H*	
5. Polished Aggregate (Square Meters) No Severity Levels		13. Spalling of Longitudinal Joints (Meters) L, M, H*	
6. Popouts (Number) No Severity Levels		14. Water Bleeding and Pumping (Number of Occurrences and Length of Affected Pavement) (Meters)	
7. Blowups (Number) No Severity Levels		15. Longitudinal Joint Seal Damage (Meters) No Severity Levels	

* Low, Moderate, and High Severity Levels

Figure 1-3. SHRP LTPP distress symbols for continuously reinforced concrete pavements (CRCP) (SHRP 1993).

Table. Description of symbols for figure 1-3.

Distress Number	Distress Name	Symbol Description	Severity Indication	Other Indications
2	Longitudinal cracking	Vertical line at the crack site	L, M, H	Length of crack in meters; write the letter S on the map at the distress site if the crack is sealed
3	Transverse cracking	Horizontal line at the crack site	L, M, H	Number of cracks and length in meters
4a	Map cracking	Rounded plaid/fishnet pattern	No severity levels	Square meters of distress
4b	Scaling	Rounded plaid/fishnet pattern	No severity levels	Square meters of distress
5	Polished aggregate	Small diagonal lines	No severity levels	Square meters of distress
6	Popouts	Small ovals drawn at an angle	No severity levels	Number
7	Blowups	Curved line over joints	No severity levels	Number
8	Transverse construction joint deterioration	Small X's below joint at site of deterioration	L, M, H	Number
11	Patch/Patch deterioration	Shaded square over patched area	L, M, H	Square meters and number of patch; indicate on map an F if flexible or an R if rigid.
12	Punchouts	Two horizontal, parallel lines with diagonal lines drawn between the parallel lines at punchout site	L, M, H	Number
13	Spalling of longitudinal joints	Series of small, vertical circles around the joint	L, M, H	Length in meters
14	Water bleeding and pumping	Arrowhead pattern drawn on the crack	No severity levels	Number of occurrences; length of affected pavement in meters
15	Longitudinal joint seal damage	Small horizontal lines drawn over longitudinal joint	No severity levels	

Figure 1-4. Example SHRP LTPP survey form (SHRP 1993)

Distress Types and MRD Identification

Table 1-2 lists the specific distress types recognized by the SHRP LTPP distress identification manual for PCC pavements. Of these distresses, one is related specifically to a durability problem (D-cracking) and three are possibly characteristic of an MRD: spalling, map cracking, and scaling. The selection of D-cracking as a distress stands out from the other distresses in Table 1-2 in that identifying the distress type in effect presumes a durability problem. As discussed previously, many of the various MRD types can display similar manifestations. In the case of D-cracking, the observed manifestation is often associated with aggregate freeze-thaw deterioration, which suggests the possibility of misidentification if an MRD is designated as D-cracking based solely on visual appearance.

Table 1-2. SHRP LTPP distress types for PCC pavements (SHRP 1993).

Distress Type	Severity Levels	Quantity Measure
Corner Breaks (<i>JCP only</i>)	L, M, H	Number
Durability (D-) Cracking	L, M, H	Number of Slabs, Square Meters
Longitudinal Cracking	L, M, H	Meters
Transverse Cracking	L, M, H	Number, Meters
Transverse Joint Seal Damage (<i>JCP only</i>)	L, M, H	Number
Longitudinal Joint Seal Damage	None	Number, Meters
Spalling of Longitudinal Joints	L, M, H	Meters
Spalling of Transverse Joints (<i>JCP only</i>)	L, M, H	Number, Meters
Map Cracking	None	Number, Square Meters
Scaling	None	Number, Square Meters
Polished Aggregate	None	Square Meters
Popouts	None	Number/Square Meter
Blowups	None	Number
Faulting of Transverse Joints (<i>JCP only</i>)	None	Millimeters
Lane-to-Shoulder Dropoff	None	Millimeters
Lane-to-Shoulder Separation	None	Millimeters
Patch/Patch Deterioration	L, M, H	Number, Square Meters
Water Bleeding and Pumping	None	Number, Meters
Transverse Construction Joint Deterioration (<i>CRCP only</i>)	L, M, H	Number
Punchouts (<i>CRCP only</i>)	L, M, H	Number

Spalling and scaling distress manifestations can all be associated with MRD in some manner. However, in the LTPP survey procedure little additional information is sought on the specific characteristics of these distresses. For instance, the distress identification manual notes that map cracking is limited to the upper concrete surface, but seeks no other information on the pattern of cracking or on any associated staining or material deposits or exudations. Furthermore, because distress symbols are used in noting the distresses, it is difficult to get an accurate representation of cracking patterns and locations.

As with map cracking and scaling, transverse and longitudinal joint spalling are rated based only on their dimensions, with no information sought on associated crack patterns or staining. And, in assessing joint

spalling, no mention is made of the possible associated presence of D-cracking, which gives rise to the possibility that both distresses are counted in a distress survey, when in fact the joint spalling is caused by D-cracking (the severity of the D-cracking is based on the amount of cracking and the amount of loose or spalled material).

Summary of the SHRP LTPP Procedure

A review of the SHRP LTPP distress identification manual and survey procedure indicates that it is limited in its ability to identify MRD. Four distress types that are representative of MRD are noted in the manual (D-cracking, map cracking, scaling, and spalling), but there are no attempts made to collect additional information (such as staining or crack patterns) that would be useful in identifying MRD. Furthermore, the D-cracking distress noted in the manual implies a cause of the distress and not just the actual distress manifestation. These limitations were addressed in Guideline I by adding methods to describe cracking and spalling patterns and locations and descriptors for staining and exudate.

Corps of Engineers PCI Procedure

Description and Purpose

The PCI procedure was developed by the Army Corps of Engineers as a means of determining the current condition of a pavement network (Shahin and Walther 1990). Extensive work went into the development of a numerical index value that is used to represent the pavement's structural integrity and its surface operational condition based on the observed distress. The resulting index, the PCI, ranges from 0 (failed pavement) to 100 (perfect pavement) and accounts for the types of distress, the severity of the distresses, and the amount or extent of the distresses. The associated effects of these factors are combined into a composite PCI value through established "weighting factors" so that it more accurately reflects the overall performance of the pavement (Shahin and Walther 1990).

The PCI procedure is intended primarily for network-level pavement management purposes, not only in documenting the current condition of the pavement but also in developing prediction models to forecast future pavement condition (Shahin and Walther 1990). However, the methodology is sufficiently comprehensive and flexible that the PCI procedure can also be used in project-level analyses.

Survey Procedure

The PCI procedure first requires that the pavement section be divided into smaller subsections for inspection purposes. These smaller subsections are called sample units and, for concrete roads, the recommended sample unit size is 20 ± 8 slabs (if the joint spacing of the pavement is greater than 7.6 m, imaginary joints less than 7.6 m apart are assumed) (Shahin and Walther 1990). The sample unit size (i.e., number of slabs) should be selected as a matter of convenience; for example, if a pavement section consists of 200 slabs, then 8 sample units consisting of 25 slabs provide a convenient means for inspection. For data management purposes, each of the individual sample units is numbered from 1 up to the total number of sample units in the section; in the previous example, the sample units would be numbered from 1 to 8.

The type of analysis (network- or project-level) will determine the number of sample units to be inspected. For project-level analysis, a greater number of sample units should be inspected because of the need to accurately assess pavement performance and to prepare accurate work plans and contracts. The minimum number of sample units (n) that must be surveyed in a project-level analysis in order to obtain a reasonable estimate of the pavement section's PCI is determined from the following formula (Shahin and Walther 1990):

$$n = [N * s^2] / [(e^2/4)(N - 1) + s^2] \quad (1-1)$$

where:

n = Minimum number of sample units to inspect.

N = Total number of sample units in the pavement section.

s = Standard deviation of the PCI between sample units (generally assumed to be 15 for PCC pavements).

e = Allowable error in the estimate of the section PCI at 95 percent confidence (generally set to 5).

When the total number of sample units (N) is less than 5, then it is recommended that all of the sample units be surveyed.

Once the minimum *number* of sample units has been determined, the next step is to determine which *specific* sample units should be inspected. This is accomplished by a stratified sampling procedure that first determines the sampling interval (i) as follows:

$$i = N / n \quad (1-2)$$

where:

i = Sampling interval.

n = Minimum number of sample units to inspect.

N = Total number of sample units in the pavement section.

The first sample unit to be surveyed is selected at random between sample unit 1 and the sampling interval i (e.g., if $i = 3$, then the first sample unit to be surveyed is randomly selected between 1 and 3). After the first sample unit is selected, the next sample units to be surveyed are selected by adding the sampling interval to the first sample unit number; for example, if the first sample unit is 2 and the sampling interval is 3, then the sample units to be inspected would be 2, 5, 8, 11, and so on up to the total number of sample units (N).

As with the LTPP procedure, the inspection is conducted by the survey crew walking over the length of each sample unit and recording the distress found in each slab. An example form that can be used to conduct the survey is shown in figure 1-5. This form is for one sample unit only; additional forms are required for each sample unit within the section. The pavement slabs in the sample unit are drawn in the space on the right portion of the form. However, according to the procedure, the incidence of distress is not drawn in, but rather is noted to occur in the slab using the number code shown in figure 1-5. Thus, it is not clear exactly where on the slab the distress is occurring.

Distress Types and MRD Identification

The specific distress types recognized by the PCI procedure are listed in table 1-3. This table shows that the distress is recorded on a slab-by-slab basis, and that actual distress quantities are not obtained (e.g., linear measures of cracking); rather, only the presence of the distress is noted. Furthermore, if more than one of a particular distress type occurs within a slab, only the highest severity is recorded so the presence of a less severe distress of that type is ignored. The data obtained in this fashion are useful in gaining an overall perspective on the performance of the pavement, but the end result is that true distress quantities are not collected.

Although never specifically stated, the PCI procedure is geared to jointed concrete pavements. No mention is made regarding how a CRCP should be surveyed (since the survey is based on a slab-by-slab evaluation procedure). Furthermore, none of the distresses listed in table 1-3 is specifically for CRCP structures; in fact, the punchout distress listed in table 1-3 refers to punchout structural failures occurring between a transverse joint and a nearby crack.

The PCI procedure recognizes one manifestation directly attributable to MRD (D-cracking), clumps three other distress manifestations consistent with MRD together as a single distress type (scaling/map cracking/crazing), and breaks out another distress manifestation common with MRD (spalling) into two varieties (corner and joint). As with the SHRP LTPP procedure, the designation of the D-cracking distress represents a diagnosis, which could lead to the misidentification of the distress. And the grouping of the scaling/map cracking/crazing

Figure 1-5. Example PCI sample unit inspection form (Shahin and Walther 1990).

Table 1-3. PCI distress types for PCC roads (Shahin and Walther 1990).

Distress Type	Severity Levels	How Rated/Recorded
Blowup/Buckling	L, M, H	Per slab basis/Highest severity recorded
Corner Break	L, M, H	Per slab basis/Highest severity recorded
Divided Slab	L, M, H	Per slab basis/Highest severity recorded
Durability (D-) Cracking	L, M, H	Per slab basis/Highest severity recorded
Faulting	L, M, H	Per slab basis/Highest severity recorded
Joint Seal Damage	L, M, H	Per sample unit/Average condition recorded
Lane/Shoulder Drop-Off	L, M, H	Per slab basis/Average condition recorded
Linear Cracking	L, M, H	Per slab basis/Highest severity recorded
Patching, Large (> 0.45 m ²)	L, M, H	Per slab basis/Highest severity recorded
Patching, Small (< 0.45 m ²)	L, M, H	Per slab basis/Highest severity recorded
Polished Aggregate	None	Per slab basis
Popouts	None	Per slab basis
Pumping	None	Per slab basis
Punchout (<i>non-CRCP</i>)	L, M, H	Per slab basis/Highest severity recorded
Railroad Crossing	L, M, H	Per slab basis/Highest severity recorded
Scaling/Map Cracking/Crazing	L, M, H	Per slab basis
Shrinkage Cracks	None	Per slab basis
Spalling, Corner	L, M, H	Per slab basis/Highest severity recorded
Spalling, Joint	L, M, H	Per slab basis/Highest severity recorded

manifestations into a single distress appears to have been done as a matter of convenience and perhaps in recognition of their perceived similar effect on pavement performance; however, in grouping these manifestations together, the PCI procedure makes no attempt to distinguish between the unique causes of each manifestation. Furthermore, the inclusion of two spalling manifestations appears to complicate the identification of possible distress mechanisms, and no attempt is made to distinguish spalling associated with D-cracking with other joint spalling. Finally, as with the SHRP LTPP procedure, no attempt is made to capture associated distress manifestations (crack patterns, staining, material depositions) that may be useful in identifying MRD.

The primary goal of the PCI survey is to obtain a numerical index value representing the current condition of the pavement, and this is accomplished through deduct charts used to assess the relative effect of each distress type on the performance of the pavement. Thus, specific charts for each distress type are needed for this evaluation, meaning that only the distresses listed in table 1-3 are assumed to affect pavement performance. In order for other distress types to be included, deduct charts would have to be developed and incorporated into the procedure.

Summary of Corps of Engineers PCI Procedure

The PCI procedure is intended more for network-level pavement condition surveys where the presence of the various distress types is important in assessing the overall condition of the pavement section. The procedure incorporates detailed sampling plans so that representative measures of performance are

obtained. However, the PCI procedure only recognizes three distress types or manifestations consistent with MRD (D-cracking, scaling/map cracking/crazing, and spalling) and makes no attempt to collect additional information that may be useful in identifying mechanisms and causes of distress. Furthermore, the procedure is geared toward jointed concrete pavements and no guidance is provided on surveying CRCP structures.

ACI Guide for Making a Condition Survey of Concrete in Service (ACI 1992b)

Although not a detailed pavement survey procedure, this ACI document does provide some useful information in evaluating the performance of a concrete pavement. For example, it provides a comprehensive checklist of information to be considered during a condition survey. Although the checklist is generic to all concrete structures and not all information given in the document will be needed for each survey, a review of the various items is useful to ensure a thorough survey. A listing of the major heading items found in the checklist are summarized in table 1-4 (the complete checklist contains many sublevel items that are not repeated here).

The ACI document also provides a detailed listing of concrete distress manifestations, including a description of the manifestation and photographs of the distress. The listing is very comprehensive as it includes distresses for all concrete structures, and introduces many distress types not normally encountered in a pavement distress identification manual. It also includes specific distresses that fit into the MRD category; these distress items, and their associated definitions, are (ACI 1992b):

- Craze cracks – fine, random cracks or fissures in a surface of concrete.
- D-cracking – a series of cracks in concrete near and roughly parallel to joints, edges, and structural cracks.
- Pattern cracking – fine openings on concrete surfaces in the form of a pattern resulting from a decrease in volume of the material near the surface, an increase in volume of the material below the surface, or both.
- Efflorescence – a deposit of salts, usually white, formed on a concrete surface, the substance having emerged in solution from within the concrete and subsequently precipitated by evaporation.
- Exudation – a liquid or viscous gel-like material discharged through a pore, crack, or opening in the surface of concrete.
- Scaling – local flaking or peeling away of the near-surface portion of hardened concrete.
- Spalling – a concrete fragment, usually in the shape of a flake, detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the large mass.
- Discoloration – departure of color from that which is normal or desired.

Table 1-4. Simplified ACI survey checklist (ACI 1992b).

CHECKLIST ITEMS
<i>1. Description of structure or pavement</i>
1.1 Name, location, type, and size
1.2 Owner, project engineer, contractor, when built
1.3 Design
1.4 Construction
1.5 Photographs
1.6 Sketch Map
<i>2. Present condition of structure</i>
2.1 Overall alignment of structure
2.2 Portions showing distress

2.3 Surface condition of concrete
2.4 Interior condition of concrete
3. <i>Nature of loading and detrimental elements</i>
3.1 Exposure
3.2 Drainage
3.3 Loading
3.4 Soils
4. <i>Original condition of structure</i>
4.1 Condition of formed and finished surfaces
4.2 Defects
5. <i>Materials of construction</i>
5.1 Hydraulic cement
5.2 Aggregates
5.3 Mixing water
5.4 Admixtures
5.5 Concrete
6. <i>Construction practices</i>
6.1 Storage and processing of materials
6.2 Forming
6.3 Concreting operation

Other than the D-cracking definition (which, like the other procedure reviewed, represents a distress diagnosis), these definitions represent an attempt to record and classify many of the visible distress manifestations associated with MRD. These are used as the basis for visible distress items included in the field survey guidelines.

Applicability of Available Distress Survey Procedures for the Assessment of MRD Affected Pavements

Although a variety of distress survey procedures and distress identification manuals have been prepared over the years, none fully serve as a comprehensive document for the field survey and evaluation of concrete pavements with MRD. Two primary distress survey procedures—the SHRP LTPP procedure and the COE PCI methodology—are lacking the ability to identify MRD manifestations. A guide produced by the ACI provides some useful information on data elements needed for a distress survey and on some MRD-associated distress manifestations, but does not describe survey procedures. Together, however, these documents provide the foundation for what is required in an MRD-related survey.

Procedures Adopted for a Distress Survey of MRD Affected Concrete Pavement

The goal of an MRD pavement survey is to collect all information needed to assist in the identification of the type of MRD. Although the MRD survey may provide strong indications as to the causes of the MRD, it is recommended that final determination be made based on the results of extensive laboratory testing and evaluation. Thus, it is imperative that the field survey collects all information necessary for later testing and evaluation.

Based on information obtained from the distress survey procedures previously presented, and in anticipation of the needs for a detailed MRD survey, the approach presented in Guideline I contained in

Volume 2 of this Final Report was developed. Since the method is presented in its entirety in Volume 2, it will not be duplicated here. Instead, only a brief summary of some salient points is provided below.

Records Review

The first part of the MRD survey should begin with a preliminary records review. This should include all information needed to adequately identify and assess the pavement in the field, such as project location, structural design (slab thickness, base type, joint design, subgrade support, and so on), mix design and materials information (including aggregate type, aggregate source, and cement type), and traffic data (traffic volumes, truck volume composition). Forms for recording this information are provided in Guideline I.

Shoulder Survey and Sample Unit Selection

Immediately before conducting the distress survey, a shoulder survey of the entire project should be conducted in both directions when both directions represent the same project (structural design, mix design, construction, and so on). The purpose of the shoulder survey is to assess the overall performance of the project and to assess the extent and uniformity of MRD. Any directional differences in performance or distress manifestations should be noted by location to assist later in the selection of sample units.

After the shoulder survey has been conducted, the project must be laid out so that sample units may be selected for survey. Sample units are intended to be representative of the entire project and are used so that a complete evaluation of the entire project (which typically is several kilometers long and would require a substantial amount of time and effort to evaluate) is not necessary.

The number and location of the sample units will depend largely on the overall condition of the pavement. This is because the purpose of the field evaluation is to collect data and material samples necessary to aid in the identification of the type and causes of MRD. In other words, a problem has already been identified in the pavement and a detailed evaluation is being conducted to determine the causes of the distress. This is in contrast to a network-level pavement survey in which statistical sampling procedures are employed requiring that typical, representative sample units be evaluated to make statistically valid assessments of overall pavement performance.

The results of the shoulder survey are used in the selection of the location and number of sample units to be surveyed. Generally, if the shoulder survey indicates that the extent and severity of MRD are fairly uniform throughout the length of the project, then a single sample unit selected at a representative location is believed to be sufficient. If, however, the shoulder survey indicates a range of MRD severity levels, or areas where there is no distress and other areas where there is severe distress, then surveying sample units in each of these areas may be appropriate to determine why one area is performing differently than another.

Pavement Distress Survey

It is recommended that the distress survey be conducted in general accordance with the procedures and distress definitions found in the SHRP LTPP distress identification manual (SHRP 1993) with several modifications to address the previously described limitations. These include the removal of D-cracking as a distress manifestation and the addition of procedures to provide a detailed description of the location and patterns associated with cracking and spalling. Additional descriptors were added for staining and exudate.

However, there is no need to draw every crack associated with the MRD acting on a pavement. Such documentation is time-consuming and does not contribute to the identification of the distress. Instead,

the general area of MRD will be noted and the physical manifestations characterized on specific MRD survey forms.

The limits of the sample unit are first identified and laid out; the distress survey then proceeds from the beginning of the project. All distresses occurring within the sample unit are noted and, where appropriate, drawn approximately to scale on the general distress survey form.

The primary distress survey should generally be conducted in only one lane (e.g., the outer lane of a multi-lane facility or one direction of a two-lane roadway). However, in viewing the adjacent lane from the primary lane, any distresses that are clearly visible should be marked on the distress forms to provide some overall indication of the presence of distress in those lanes.

Although identification of the MRD will not be made during the field surveys, attempts should be made to characterize to the greatest extent possible the visible signs of MRD, including:

- Cracking pattern (location, orientation, extent, crack size).
- Staining (location, color).
- Exudate (presence, color, extent).
- Scaling (location, area, depth).
- Presence of vibrator trails.

The typical MRD cracking pattern will also be recorded during the field surveys. This will be done for only one typical slab to illustrate a schematic of the prevailing crack pattern. However, to facilitate the field survey, the MRD cracking is not drawn on the distress survey forms (but the general area of distress is noted). Detailed quantities of MRD are entered on another form.

As mentioned, several different data collection forms were developed to record observed distress manifestations. These include:

- A general information form that describes the location and characteristics of the project. Most of this form can be completed prior to visiting the field site.
- A field distress survey form similar to that used in the SHRP LTPP program (SHRP 1993). All pavement distresses are to be recorded on this form, in addition to the *general area of occurrence* of the MRD on the pavement.
- Two additional forms were developed to help characterize the MRD. One of the forms provides space to draw the typical MRD pattern observed on a single slab, including the crack pattern and the presence of staining and any exudation. The second form is then used to categorize the MRD distress in terms of its cracking pattern (location, extent, size), staining (location, color), material deposits or exudation (presence, color, extent), and scaling.

Photo/Video Documentation

At the conclusion of the distress surveys, a complete photo or video summary of the sample unit is performed. The purpose of this is to fully and completely document the performance of the pavement, as well as to record the prevailing foundation and drainage characteristics of the roadway. Because some MRD “disappear” under poor lighting conditions, the photo documentation should be obtained in such a manner that the distresses are clearly visible.

Field Chemical Testing

Several test procedures are available that might be useful in the field evaluation of pavements suspected of suffering ASR. These tests use chemicals that are applied to a pavement sample in the field to

indicate whether ASR is present. Two tests for the determination of ASR distress are described below; currently, no other chemical tests are available for the preliminary identification of MRD in the field.

Gel Fluorescence (Uranyl Acetate) Test

The gel fluorescence test was first developed as a laboratory procedure at Cornell University and was later modified for field use (Natesaiyer and Hover 1992). In this test, the gel-like reaction products of the ASR reaction are identified by staining them with uranyl ions; these ions have a characteristic bright greenish-yellow fluorescence when viewed under UV light. Other normal compounds in concrete are not typically stained by the uranyl ions. The test can be conducted in the field, but proper guidelines must be followed because of the radioactivity of the uranyl acetate compound. The conduct of this testing is included as an annex to ASTM C 856.

Although preliminary results with this test have been encouraging, a positive identification by this technique does not necessarily mean that ASR is present (Farny and Kosmatka 1997). For example, some aggregates fluoresce naturally and some carbonated concrete can show a mild fluorescence in the areas of carbonation (Natesaiyer and Hover 1992). Thus, the test is not intended to be a stand-alone method for diagnosing ASR, but rather an ancillary test to more definitive laboratory testing (Farny and Kosmatka 1997). Furthermore, the test requires experienced and knowledgeable technicians for correct interpretation of the findings.

Additionally, because of the perceived hazards associated with uranyl acetate and the disposal of the stained concrete, this test is not currently recommended for use in field surveys of pavements with potential MRD problems.

Sodium Cobaltinitrite/Rhodamin B Test Method

In recent years, a test developed at the Los Alamos National Laboratory has become available for the field identification of ASR gel (Guthrie and Carey 1997). The advantages of the test procedure are that it uses non-radioactive chemicals (sodium cobaltinitrite and rhodamin B) to stain ASR reaction product, the stains are specific for ASR gel, and the staining is visible in normal light.

Due to the recent introduction of this test, little information is currently available regarding its effectiveness to make positive identification of ASR in the field. It does promise to be simpler and safer to run, but additional laboratory testing is required to confirm the both presence and extent of ASR. Thus, while consideration should be given to the further evaluation of this technique, it is not recommended as part of an MRD field survey at this time.

[Assessing the Likelihood of MRD](#)

Once the visual assessment is completed, the analyst is directed to a flowchart to assess whether the observed distress is likely the result of an MRD. The flowchart asks questions regarding the nature of the cracking, whether staining and/or exudate are present, evidence of expansion, or whether the distress is progressive. An affirmative answer suggests that an MRD may be at work, thus a laboratory analysis should be conducted necessitating the acquisition of samples.

[Review of Sampling and Handling Methodologies](#)

This section summarizes currently accepted and recommended practices for obtaining and handling samples of existing PCC pavements for the purpose of diagnosing MRD. It also discusses techniques for retrieving, handling, and storing the specimens in a manner that both maintains the physical integrity, and chemical composition and preserves the nature and structure of the microstructure present.

The importance of effective sampling and testing procedures should not be overlooked. The basic principles and approaches on the topic have been developed and disseminated by the American Society for Testing and Materials (ASTM) Committee E11 on Statistical Methods (ASTM E 105; ASTM E 141) and others (ASTM 1948; Tanner and Deming 1949; Slohim 1960; Montgomery 1985; Tanner 1961; Cochran 1963; Bicking 1964; Bicking 1965; Duncan 1967; Visman 1969; ASTM 1973; ASTM 1975; SHRP 1990).

Additional information on sampling procedures for concrete and concrete ingredients are found in various ASTM standards (C42, C823 and C 856) and other handbooks and manuals such as those by the Waterways Experimental Station (WES) (1949), ACI (1981), and the United States Bureau of Reclamation (USBR 1975). Useful references to existing ASTM standards dealing with probability sampling concepts and the sampling of various concrete component materials and hardened PCC pavements are presented in table 1-5.

Table 1-5. ASTM sampling standards for concrete and concrete-making materials.

Designation	Standard
C 172	Practice for Sampling Freshly Mixed Concrete
C 183	Method for Sampling and Testing of Hydraulic Cement
C 311	Method for Sampling and Testing Fly Ash for Use as an Admixture in Portland Cement Concrete
C 702	Method for Reducing Field Samples of Aggregate to Testing Size
C 823	Practice for Examination and Sampling of Hardened Concrete in Construction
D 75	Practice for Sampling Aggregate
D 345	Methods for Sampling and Testing Calcium Chloride for Roads and Structural Applications
D 3665	Practice for Random Sampling of Construction Materials
E 105	Practice for Probability Sampling of Materials
E 122	Practice for Choice of Sample Size to Estimate the Average Quality of a Lot or Process
E 141	Practice for Acceptance of Evidence Based on the Results of Probability Sampling

As few of these practices are not directly applicable to sampling concrete pavements for the investigation of MRD, they will thus not be discussed in detail in this report. Instead, the information presented in this section will focus on the unique sampling requirements to obtain PCC specimens for subsequent laboratory testing and analyses to provide accurate and representative indications of the presence of MRD.

Approach Adopted for Sampling and Handling MRD Affected Pavements

This section discusses the preparation activities, sampling and handling procedures, and shipping procedures recommended for use when sampling a concrete pavement suspected of being affected by MRD. The sampling procedures presented are designed only for the purpose of identifying MRD, and are not to be used in the course of construction quality control.

General Approach to Sampling

ASTM C 42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*, governs the retrieval of cores or the sawing of beams from hardened concrete. However, this standard does not mention how to decide at what point or points such specimens should be secured. ASTM C 823, *Standard Practice for Examination and Sampling of Hardened Concrete in*

Construction, describes this, as well as many other aspects of sampling hardened concrete, in considerable detail. Other sources of information include Abdun-Nur (1970) and ACI 214, *Evaluation of Results of Tests Used to Determine the Strength of Concrete*. It is noted that standardization of the sampling process is difficult because it involves a great deal of judgment that is often difficult to standardize.

If the purpose of sampling hardened concrete is to assess overall properties and variability, it should be done randomly and objectively. The selection of samples in such cases should not be biased by procedures that intentionally select either the best or poorest materials. However, sampling in the course of investigations of MRD are distinctly different from quality control/assurance testing in that samples should be obtained to investigate unusual or extreme conditions or features that will aid in the identification of the causes of distress or failure of the concrete. It is this type of sampling approach that has been adopted.

In hardened concrete being analyzed for MRD, specimens could be obtained by random sampling from within a panel to determine the variability of the development of MRD within the panel. Or sampling could be concentrated in areas of known or suspected MRD development in order to characterize the conditions that have led to that development. The latter approach is recommended due to the time and expense involved in analyzing concrete in the laboratory. Thus it is extremely important that sampling in the field (including the development of an appropriate sampling plan) is not a simple matter and must not be delegated to the untrained or careless, or relegated to a laborer for the sake of convenience (Abdun-Nur and Poole 1994).

The sampling plan must be selected with consideration of the results of field condition surveys that have been discussed previously in this report. The visual inspections and review of the construction records will indicate whether or not the concrete is believed to be in similar condition and of similar quality. The sampling plan must also reflect the full prior knowledge of the laboratory-testing program that will be performed, as is discussed later in this report. For the purpose of an MRD investigation, evidence of distress will be obtained from the results of the visual assessment conducted in accordance with the guidelines. Specimens for use in the laboratory investigation should be selected based on “targeted sampling,” since they are being used solely to characterize the type and extent of distress.

Data collected during the field survey will be used in the selection of coring locations. For example, information detailing the location and characteristics of the project should be reviewed by the person(s) developing the sampling plan as well as by the persons that will be executing the sampling plan prior to their visit to the field. This information should include a detailed sketch or drawing of typical observed MRD, including the crack pattern and the presence of staining and any exudation. It may also include categorization of the MRD distress in terms of its cracking pattern (i.e., location, extent, and size), staining (i.e., location and color), and material deposits or exudation (i.e., presence, color, and extent).

The first step in determining the number of coring sites for a given project is to use the results of visual survey to assess the overall continuity of MRD within the project length. If the distress is uniformly distributed over the entire project length, one or two coring sites are considered sufficient to accurately characterize the type and extent of MRD. If observed distress severity and/or extent vary along the project length, at least one coring site must be selected for each observed condition level.

Coring sites should always correspond to the sample units inspected during the visual assessment. In this way, the detailed crack maps can be used to initially select specific core locations. An adequate number of cores must be obtained to represent the condition of the pavement, yet the number must be restricted due to practical restraints on resources and the specific needs of the investigation. In the guidelines developed in the course of this study, each PCC pavement coring site is 150 m long. It is recommended that a minimum of four cores be obtained per coring site. Specific details concerning recommended core locations are presented in the guidelines contained in Volume 2 of this Final Report. The following presents a brief summary of the procedures.

The quantity or number of concrete samples and the dimensions of the pieces retrieved in the sampling operation must conform to the stipulations of the tests to which the samples will be subjected. In some instances, two or more tests may be performed on a single sample, provided that the preceding tests do not modify the properties of the concrete to be evaluated by subsequent tests as described later in this report. One example of acceptable successive testing is the determination of air void content by linear traverse or modified point count followed by petrographic examination. An unacceptable sequence would be to stain a specimen using uranyl acetate and then attempt to analyze it using the scanning electron microscope (SEM).

The recommended coring locations are selected to provide more than enough specimens for analysis while preventing the acquisition of too many specimens, which leads to waste. If the distress in a JCP is concentrated at the joints, a minimum of four core samples (recommended 150-mm diameter) is required if the pavement is doweled. Two of the cores are positioned directly over the joint, with the center of the 150-mm core barrel offset 50 mm from the middle of the joint. In this way, the core can be cut across the joint in the direction of traffic to analyze the concrete at the interface to a distance of 125 mm from the joint. One of these two cores is obtained directly over a dowel bar located between the wheel paths to assess the corrosion of embedded steel. The second is obtained between dowel bars in an area affected by the MRD. The third core is obtained at a corner location, located approximately 0.3 m from both the transverse joint and outside lane stripe. This core will be used to assess the extent of deterioration away from the joint, and thus may or may not be in an area visibly affected by MRD. The final core is positioned in a center slab location free of observable distress to determine if distress is present at a microstructural level even though it is not yet visible on the surface.

For a JCP suffering from MRD that is not concentrated at the joints, five cores are required. The first four are located the same as previously described. The fifth core will be obtained in the slab interior over visible deterioration to assist in identifying its cause. This deterioration may be a crack (as is common in map cracking), spall, or delamination. In some cases, this core may be pulled from deteriorated vibrator trails. It is noted that, as stated previously, the fourth core should be taken from a visually non-distressed area. In some severe cases of MRD, a non-distressed area might not be visible, in which case the core can be taken from a deteriorated area or not at all.

A similar pattern was developed for CRCP. One notable difference is that instead of obtaining a core through a dowel bar, it instead should be taken through the reinforcing steel at a transverse crack. All other cores should be obtained between the steel, if possible. If corrosion of embedded steel is the specific distress being investigated, additional cores can be obtained through the steel at different locations at the discretion of the investigator.

It is noted that these recommendations present the *minimum* number of cores needed for the evaluation. In some instances, additional cores would be useful, particularly if certain unusual features were observed. Field crews should be instructed to take additional samples if unusual features are observed, carefully noting the location and the feature of interest.

All core samples taken should include the full thickness of the concrete because the development of cracking, deterioration of the cement paste, progress of cement-aggregate reactions, and other features may vary significantly with depth. The diameter of core specimens for MRD identification is not critical as long as it is sufficient to provide a representative section of the concrete structure once it is prepared for examination. In general, the use of 150-mm-diameter cores is preferable because it provides more material for evaluation and archiving, although 100-mm-diameter cores may suffice. Cores that are intended for use in petrographic examination must be unaltered cores that have not been subjected to strength testing or other destructive processes.

Pavement surfaces should be marked at the coring location to indicate the direction of traffic before coring or sawing. The markings should be clearly visible and easily interpreted after the coring operation is complete. Procedures that cause mechanical or thermal damage to cores should not be used in the retrieval of specimens and samples must be protected from contamination, damage, and other processes



that might change the character of the material being examined or tested. Possible sources of contamination include other layers at the same site as well as materials carried by coring rigs and other equipment from other sites. It is desirable that the surface of all cores or beams be rinsed with fresh water to remove coring or cutting slurry prior to wrapping or packing for transport and/or shipping.

Information concerning the location, manner of retrieval, and field condition of all hardened concrete samples should be recorded for future reference. Self-adhesive labels that clearly identify the core (including the coring site, slab number, core location, and date of coring) should be prepared and affixed to each sample. A sample hardened concrete sampling log sheet and sample identification label have been provided in the guidelines contained in Volume 2 of this Final Report.

Handling and Storage of PCC Field Samples

Simply establishing sampling plans, methods, and procedures are not enough to ensure that proper, undamaged samples will reach the laboratory for testing. For example, Wills (1964) has shown the undesirable effects on concrete properties caused by the use of bags contaminated with sugar, flour, or treated with chemical preservatives. Although many problems concern fresh concrete specimens, proper protection, packaging, and care of hardened concrete cores in the field and during shipping are still important.

In general, samples must be properly identified and shipped in clean, strong containers. If the moisture content of the sample is important, the container must be moisture-tight. Containers should always be clean and free of any potential contaminants. Care should be taken to prevent damage to concrete specimens during transport and shipping. Intact cores or large core fragments can generally be transported from the field and shipped over short distances without damage when wrapped in plastic bubble wrap packing material and placed in appropriately sized plastic cylinder molds with lids. Similarly, sawed beams can also be wrapped in plastic bubble wrap packing material and shipping tape and placed in protective tubs or containers for transport from the field and shipping over short distances. Specimens that will be shipped by common carrier should be afforded additional protection from damage. One other interesting consideration is that specimens obtained in an area under United States Department of Agriculture quarantine must only be shipped in accordance with the relevant regulations if contaminated soil is present. Specific recommendations are provided in the guidelines provided in Volume 2 of this final report.

There are generally no limits on the duration of storage for hardened concrete specimens, provided that the desired storage conditions (if any) are maintained with respect to specimen moisture and temperature, exposure to atmospheric conditions, and so on. However, the condition and composition of the specimen must closely represent the condition of the field concrete at the moment of retrieval. Significant delays in testing may reduce the usefulness of the test results.

Summary of Background for Field Distress Survey, Sampling, and Handling Procedures

This section of the report has reviewed the background information and quickly summarized the approach presented in the first guideline on field distress survey, sampling, and handling procedures. Several pavement distress identification and survey procedures were reviewed to determine their applicability for use on MRD pavements. Current procedures provide a basis for evaluating MRD affected pavements, but are lacking in ways of identifying and collecting distress manifestations specific to MRD. Based on the current procedures and recognizing the specific needs for characterizing MRD, a framework for the components of an MRD survey was developed and presented in this chapter. A review of field chemical staining tests was also conducted, but it is recommended that no additional testing should be included in the guidelines at this time.

A review of procedures for obtaining and handling field samples of PCC for use in identifying the presence of MRD in PCC pavements was also presented. The procedures adopted in the guideline were

then reviewed. The section describes a systematic approach for determining the numbers of samples required and appropriate locations from which to retrieve those samples, as well as techniques for retrieving, handling, and storing the specimens in a manner that maintains the physical integrity and chemical composition of the material. Compliance with these guidelines will help to ensure that subsequent laboratory tests and analyses provide accurate and representative indications of MRD presence or potential.

3.2 Background for Laboratory and Data Interpretation Procedures

This section of the report provides the background used in the development of the second guideline that includes laboratory methods and data interpretation procedures. It provides a review of the available procedures followed by a summary of the methodology adopted in this work.

Review of Laboratory Procedures for Investigation of Concrete Deterioration

When MRD is suspected of playing a role in the premature deterioration of concrete, laboratory tests are almost always required to help understand the underlying mechanisms at work. In reviewing the various types of MRD, it is clear that the distress mechanisms involve physical and/or chemical processes that occur between the concrete and its environment. These processes ultimately lead to changes in the concrete microstructure, which may in turn affect the durability of the concrete. The relationship between material characteristics and microstructure is not unique to concrete. The study of material microstructure forms the basis of materials science and engineering. In the same vein, the typical laboratory methods used to characterize concrete's microstructure are the same as those used to characterize the microstructure of other materials.

Common References, Standards, and Practices

A number of reference materials exist that provide an excellent presentation of the current body of knowledge related to methods of performing diagnostic tests on concrete. Walker (1992) developed an excellent summary of the fundamental methods of concrete analysis, including chapters on equipment and materials, sample documentation, preparation of specimens, and the observation of cracks, voids, and paste using the stereo and petrographic optical microscopes. In addition, other concrete constituents and parameters are discussed including mineral admixtures and w/c determination. AAR is discussed and numerous photographs and micrographs are included to illustrate the different types of distresses. Various types of optical microscope methods are presented, including a section on fluorescence microscopy techniques. Numerous color pictures of various concrete specimens are included and overall, detailed steps on the fundamental tests for concrete analysis are presented.

Roy et al. (1993b) published a detailed description of the fluorescence microscopy method, including numerous micrographs showing various features as seen in fluorescence microscopy and the effects of poor sample preparation. Detailed instructions on the method of gathering and the interpretation of data are included. The textbook by St. John et al. (1998) is an excellent reference that discusses concrete petrographic techniques. It provides information and references along with details of specific techniques. The text also discusses defects in, and deterioration of, concrete and provides a wide range of micrographs illustrating common microstructural features. Also, details of optical and chemical properties of phases found in concrete are presented.

A large number of ASTM standards address laboratory practices relative to concrete. In general, three of these standards directly address the examination of hardened concrete and should serve as a basis for any laboratory examination of concrete exhibiting MRD. The applicable standards are ASTM C 457 *Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*, ASTM C 823 *Practice for Examination and Sampling of Hardened Concrete in Constructions*, and ASTM C 856 *Practice for Petrographic Examination of Hardened Concrete*. One important impact of these

standards is in the case of litigation where the credibility of laboratory results will be based in part on adherence to these accepted standards. Unfortunately, as detailed procedures, these ASTM standards often fall short, forcing the analyst or engineer to make decisions about methods without sufficient background information.

The scope of ASTM C 823 is as follows: "This practice outlines procedures for visual examination and sampling of hardened concrete in constructions. Reference is made to the examination and sampling of concrete in prefabricated building units, pre-cast products, and laboratory specimens." This practice presents a procedural plan for examining concrete that includes defining the objective, stating clearly the purpose, and defining the scope of the examination. The objective is usually the same; try to obtain information that can describe the condition of the concrete or the construction. However, in forming a clear picture of the objective, also keep in mind other constraints such as time, budget, or the interest of another party such as a contractor. In this latter case, sampling should be done judiciously in case disagreements turn into legal action. The purpose of the investigation can vary widely, including predication of service life, identification of distress, assessment of performance, or support for legal action. The type of sampling required for each purpose may vary. For example, MRD may be identified in a broken section removed by a shovel. Scientific evidence of the same distress for court action would require extensive coring and rigorous adherence to sampling statistics. The scope of the examination may be limited to one panel or 10 miles of a highway. In either case, the sampling approach required varies. It also discusses methods of performing a Preliminary Investigation indicating the need to verify the existence of a problem, describe its nature, and estimate its extent. The remainder of ASTM C 823 describes considerations and procedures for sampling under various conditions. Given the wide variety of possible Procedural Plans, ASTM C 823 should be referred to based on the specifics of each examination required.

ASTM C 457 is the standard method describing the procedure for determining the hardened air content of concrete. It results in calculated values of specific surface, frequency, spacing factor, and paste-air ratio for the air-void system of the concrete. The test method discusses equipment, procedures, and calculations for making these measurements. Recommendations of minimal sample size are provided. Note that these sampling requirements may affect the size or number of core samples required from a pavement. This method does not discuss treating the case of an air-void system filled with secondary deposits. This may be an important factor in concrete performance. It is recommended that anyone performing ASTM C 457 report both the original air content and an estimate of existing air content after infilling. This is a more tedious analysis that requires multiple analyses of the specimen. During the first examination, the analyst marks the air void at its original edge and in the second examination, the analyst marks at the edge of the filling rim, skipping completely filled voids. This is an effective way to quantify secondary deposits in air voids. Also ASTM C 457 does not produce a measure of the air-void size distribution. This characteristic is important because two concretes with the same spacing factor may have quite different freeze-thaw resistance, depending on the size and relationship of the air voids to each other (Hover 1993).

ASTM C 856 outlines many of the procedures required for the petrographic examination of concrete. This Practice should be used as the framework of any laboratory practices developed. Included in ASTM C 856 are sections on qualifications of petrographers, purposes of examination, required apparatus, sampling, sample preparation procedures, microscopical examination, and suggested diagnostic features to examine in concrete. Also, a table is included that summarizes the diagnostic optical properties of common phases found in PCC. This table is very useful for performing petrographic microscope examination of thin sections and grain mounts. Although ASTM C 856 is very useful and comprehensive, it falls short of relating observed diagnostic features to specific MRD types. This is to be expected given the general applicability desired in a practice of this type. It is impossible to address all possible combinations of concrete failures that might occur. However, the general observations made in ASTM C 856 are sound and applicable to examination of MRD in concrete pavements. It is up to the analyst to decide how to interpret the results obtained.

To glean useful information from a laboratory analysis of concrete, proper sample preparation is a very important consideration. Once specimens are selected and prepared, the stereo optical microscope (OM) and the petrographic optical microscope are the most versatile and widely applied tools for diagnosing causes of MRD. Electron microscopy is becoming more prevalent, especially for chemical identification of reaction products and other secondary phases using energy dispersive spectroscopy (EDS). The conventional SEM (CSEM) operates with the sample in a high vacuum (10^{-6} torr), leading to accurate microanalysis but desiccation cracking of the concrete. The environmental SEM (ESEM) operates with the sample in a very low vacuum (2-20 torr), leading to poorer microanalysis but little to no desiccation cracking of the concrete. The low vacuum SEM (LVSEM) provides a reasonable compromise to the problems of the CSEM and ESEM, operating at a vacuum between the extremes of the two. X-ray diffraction (XRD) has been used in very limited instances for analysis of hardened concrete produced outside the laboratory. Analytical chemistry is used primarily to determine w/c. All of these procedures and techniques are described in the following sections.

Sample Preparation

French (1991) has presented a comprehensive review of cement petrography and detailed steps for sample preparation, analysis, and interpretation. For concrete petrography with an OM or SEM, specimens may be prepared as polished thin sections, polished plates, or freshly broken surfaces. In a thin section, a section of concrete at least 10 mm thick is cut from a slab and impregnated with low viscosity epoxy in a vacuum. The vacuum helps force the epoxy into cracks and pores to stabilize the slab and fill voids that would otherwise make microscopic imaging difficult. Often, the epoxy is colored or fluorescent to enhance the visibility of voids in a microscopic image (French 1991). The slab is reground, removing approximately 1 mm of material to ensure that no damage from the saw cut will be seen in the microstructure. Also, at this point, a second epoxy impregnation is often applied before bonding the concrete slab to a glass slide and grinding to the desired thickness. In all grinding and cutting steps, non-aqueous lubricants and coolants must be used to avoid altering the cement microstructure by water addition (French 1991).

The final thickness required is determined by the type of microscopy used and the information to be obtained. For observation by optical microscopy, a section thickness of 40 micrometers is suitable for observation of ASR while a maximum section thickness of 20 micrometers is required to observe the microstructure of the cement paste (French 1991). If the thin sections are to be analyzed in an SEM, section thickness is not as critical but should not be any less than 10-20 micrometers. If the specimen is too thin the electron beam, used to form an image in an SEM, penetrates the concrete specimen and creates unwanted radiation from the glass slide (Goldstein et al. 1992). The surfaces of thin sections are polished on a mechanical polishing wheel with decreasing diamond abrasive size.

French (1991) also described the benefits of polished plates. Polished plates are prepared for observation by reflected light optical microscopy or stereo microscopy and have the advantage of giving the petrographer a larger specimen area to analyze. Also, sections may be cut along the core length, providing the petrographer a different insight into the concrete microstructure. The surface of a polished plate must be polished as flat as possible and to the same degree as a thin section. In addition, it may be necessary to use a dye to enhance the contrast between phases. Polished plates can be used to examine the aggregate rock type, aggregate distribution, magnitude of alkali reactive aggregates and amount of reaction products, crack patterns, and the structure and distribution of microcracks. The air-void system parameters such as air content and spacing factor, along with the volume fraction of concrete ingredients, can be readily measured using the stereological techniques described in ASTM C 457.

In a more recent publication, Kjellsen (1996) discussed the need for flat, polished specimens when high-strength concrete is being examined. In particular, the author noted that care should be taken to prevent differential polishing of phases in the cement as a result of their different, respective hardnesses. Also, the author cautioned that high-strength concrete is typically less permeable than normal concrete and, as a result, epoxy will not penetrate as deeply during the impregnation step.

Broken surfaces are useful for examining aggregate and crack surfaces and the contents of voids. Also, fresh cement paste gel can often be observed on aggregates, indicative of ASR (French 1991). Due to the relief present on any fracture surface, observation must be performed using a high quality stereo OM or, preferably, an SEM. The ability of an SEM to examine surfaces at a high magnification and still maintain a high depth of field allows for the examination of fine crystallites or phases that may be recessed in crevices or cracks. Also, the ability to perform a chemical analysis of a cubic micrometer of the specimen allows for the identification of phases not clearly defined by petrography alone.

In addition to choosing the proper type of sample preparation method to see specific features, the quality of preparation can affect the information obtained. For example, studies of cement paste pore structure, a key parameter affecting strength and durability, can be performed using an SEM only if epoxy impregnation is performed properly. Pores and cracks on a polished surface will not produce a uniform electron yield, relative to the bulk of the sample. In many cases, "edge effects" are seen where edges around holes or cracks appear brighter than adjacent material of the same composition (Goldstein et al. 1992). Although this approach is more prevalent for images formed by secondary electrons (SE), images formed with backscattered electrons (BE) also exhibit this effect. In addition, material at the base of a pore may produce a high electron yield and appear as a bright island in the otherwise darker pore. When filled with epoxy, a continuous, low mass phase is created, providing a continuous dark shade in any electron image. Epoxy impregnation is equally important for OM petrography where the microscopist, or image analysis system, is looking for phases with a bright or fluorescent color, which was added to the epoxy before impregnation.

The most significant sample preparation problem for cement and concrete is the dehydration of chemically bound water. A vacuum is used to impregnate specimens with epoxy, affecting specimens for both optical and electron microscopy. However, the relatively low vacuums achieved by most impregnation systems lead to minimal or no damage to the specimen due to dehydration of chemically bound water. With the CSEM, the inner chamber is maintained at a pressure of approximately 1×10^{-6} torr, to maintain the focus of the electron beam. Given that the vapor pressure for water is approximately 4.6 torr at the freezing point, excess water will evaporate from the concrete, into the SEM, and chemically bound water will be lost, resulting in alteration of hydrated solids such as ASR reaction products and ettringite. In addition, it has been reported that solid phases in cement that have water as part of their structure may decompose *in situ* as a result of losing water to the CSEM vacuum. Efforts to dry a sample may result in the removal of water that is part of the CSH hydration product microstructure (Bergstrom and Jennings 1992). These problems can be partially overcome by using an LVSEM while maintaining a 1 to 2 torr vacuum in the chamber or completely overcome by using an ESEM with a chamber pressure in excess of 4.6 torr.

Further, vacuum dehydration of a specimen occurs as part of the process used to conductively coat a specimen for observation in a CSEM. Special dehydration steps are required beyond vacuum dehydration only when microcracking or hydrated phases are to be studied. The principal methods for drying are oven drying, freeze-drying, and acetone or alcohol replacement. Oven drying is the simplest approach. Specimens are placed in an oven at 95-105°C and dried until a constant weight is obtained. This approach is widely used (Diamond 1997) and can be readily applied to concrete specimens sampled from in-service concrete. Replacement by acetone or alcohol is also used to dehydrate concrete where the specimen is saturated in the solvent and, by diffusion, the alcohol replaces the water. Struble and others have used a process of water replacement by ethanol followed by ethanol replacement by ultra-low viscosity epoxy where the epoxy used was an ethanol miscible epoxy (Struble and Stutzman 1989; Stutzman and Clifton 1999). The impregnation was performed by immersing the specimens in the ultra-low viscosity epoxy solution and holding the solution at 2°C to impede the polymerization of the epoxy and allow it time to diffuse into the concrete matrix. This process dries the specimen and impregnates it in a single, two-step procedure. Freeze drying removes the water by initially freezing the specimen in liquid nitrogen or some other cryogenic medium.

When operating at vacuums common in an LVSEM or ESEM, the need for conductive coatings or special dehydration methods are eliminated. This is a significant advantage when analyzing concrete using an SEM.

Optical Microscopy – Petrography

Petrography is an analysis method dating back to the mid-1800's when a mineralogist named Highley published the first description of the petrographic optical microscope in 1856 (St John, 1998). From there, the science of petrography grew and became a staple of mineralogical and geological research. Petrography, in essence, is the study of light passing through a specially prepared rock or mineral specimen where the material is thin enough to allow light to pass. As it does, the light interacts with the material differently depending on whether it is crystalline (ordered) or amorphous (disordered). For crystalline materials, the light interaction differs depending upon the arrangements of atoms in the crystal structure. The resulting effects can be interpreted by a properly trained petrographer and used for identifying the material.

Concrete petrography refers to the application of petrography to the analysis of "man-made" rocks (i.e., concrete). One of the first reports of application of petrographic techniques to concrete was published by N.C. Johnson in 1915 (Erlin 1993) and related observations of the concrete microstructure to performance. Since then copious publications have been written on the subject, including textbooks (St John 1998; Campbell 1985). In 1962, Mielenz published a review of concrete petrography that remains a very informative resource as a background on the science. However, advances in instrumentation and development of new instrumentation have occurred since then and more recent reviews provide additional information. One such review was published by French (1991) and provides detailed descriptions of various diagnostic features of distressed concrete of all types. Optical microscopy has the advantages of being relatively inexpensive and also relatively reliable, assuming the microscopist has extensive experience examining cement structures (Campbell-Allen and Roper 1991). An optical microscope can be used to examine amorphous materials while other techniques, such as XRD, cannot easily quantify these phases. However, there are limitations to the optical microscope, such as a relatively low image resolution and no ability to perform chemical analysis of the specimen or its micro-constituents (French 1991). Even with these limitations, optical microscopy is the best choice for studying PCC microstructure and it plays a key role in everyday laboratory petrographic analysis.

Of special importance in the area of petrography is one relatively new technique that has been developed based on fluorescent microscopy. First proposed by Christensen (Thaulow et al. 1982), the use of UV-fluorescent dye was proposed for the determination of capillary porosity in hardened cement paste. In a later paper, Thaulow et al. (1982) used this method to estimate the compressive strength of concrete samples using fluorescence microscopy. After development, the method has become accepted by some as a measure of w/c in hardened concrete (Mayfield 1990; Jakobsen et al. 2000). Additionally, the UV-fluorescent dye clearly highlights cracks, microcracks, and air voids. Cracks or voids filled with reaction products also clearly stand out from the surrounding cement paste.

Scanning Electron Microscopy

Scanning electron microscopy and digital imaging have been widely applied to PCC, cement paste, and mortar characterization. As stated previously, the cement paste microstructure and the quality of the aggregate interface are the main areas of focus for researchers examining strength and durability of PCC. For cement paste, and for the interfacial transition zone (ITZ), the key parameters affecting strength are total porosity, pore size distribution, the degree of hydration, and the characteristics of calcium hydroxide phases within the paste. Many of these attributes have been studied in laboratory prepared specimens of PCC, cement paste, and mortar using an SEM and digital imaging, but little has been presented in the literature on the use of these techniques for routine characterization of concrete that has been in service. However, the same techniques used to study laboratory prepared specimens of PCC are applicable to the study of concrete that has been in service.

One example of the application of the SEM to characterize concrete microstructure was presented by Tamimi (1994). He developed a new two-stage mixing procedure that resulted in reduced bleeding, an increased minimum and maximum micro-hardness, and a high rate of strength development. In this study, XRD was used to identify principal phases resulting from the hydration of portland cement and quantify the abundance of these same phases as a function of hydration time. Additionally, he was able to characterize the crystallographic orientation of calcium hydroxide occurring at the aggregate-cement interface and show the tendency for calcium hydroxide to grow with a preferred orientation. In this study, an SEM was used to observe the resulting microstructures and to confirm observations made from XRD results by performing microanalysis of key phases. In a more recent paper, Tamimi (1996) used the SEM to study the aggregate-cement interface in concrete produced using the same two-stage mixing procedure and monitored the morphology change with time.

As discussed previously, the advent of the ESEM offers significant advantages for cement characterization. The ESEM allows samples to be viewed at pressures greater than the vapor pressure of water, eliminating the dehydration of concrete specimen. The ESEM has a unique electron detector that takes advantage of the gas molecules in the ESEM chamber to amplify the electron signal. The ability to observe specimens "wet" allows for specialized tests such as *in situ* hydration experiments. Most published ESEM work falls into this category. Using an ESEM, Bergstrom and Jennings (1992) demonstrated that C_3S phases younger than 16 hours, when observed under a conventional SEM, have a different microstructure compared to the same phases observed in an ESEM. In another study, Lange et al. (1991) used the ESEM to observe the early hydration of portland cement. The results indicated unique insights into the development of cement microstructure at early ages.

Analytical Chemistry

The role of the chemistry laboratory in analyzing an MRD in concrete is not always clearly defined. This is principally due to the fact that analytical chemistry serves multiple purposes. First, it can help diagnose routine problems. Also, the chemistry laboratory can provide quality control where tests in controlled environments are used to verify important measurements that are routinely performed using alternative methods. Some examples are cement content and determination of w/c , chloride concentration, and sulfate analysis. In addition, the chemistry laboratory provides a controlled environment where long-term studies can be conducted without interruption.

Analytical chemistry tests for the diagnosis of MRD can be conducted at almost any step in the evaluation process, but are often not performed at all. It should be noted that many chemistry procedures require relatively large volumes of concrete. Also, some chemical analyses take many days to complete. It is for these reasons that analytical chemistry testing should be conducted at an early stage of the analytical process. The most common application of analytical chemistry for diagnosing MRD in PCC is for determining the cement content of the hardened concrete, and indirectly, the w/c .

Recommended Laboratory Procedures for Investigation of MRD in Concrete Pavements

A key step in identifying the cause of MRD in concrete pavement is laboratory analysis of the distressed concrete. The laboratory analysis of concrete is facilitated by the systematic application of test methods specifically designed to identify MRD by searching for known symptomatic indicators, or diagnostic features. However, laboratory results are susceptible to broad interpretation, and a rigid adherence to laboratory protocol, along with the judgment of an experienced petrographer, analyst, and/or engineer is often required to avoid incorrectly diagnosing a problem.

This section provides a brief overview of the procedures described in the guideline presented in Volume 2 of this report. It also provides a brief description of commonly applied procedures and a suggested format for gathering and storing data. This procedure is intended for use by both engineers and analysts where an engineer is defined as the person presenting the problem to the laboratory and the analyst is

the laboratory personnel charged with diagnosing the concrete problem. In many cases, this may be the same person, but often they are members of a team of people that need to communicate clearly to arrive at a cause, or causes, for the suspected MRD. In this sense, both engineers and analysts must have a clear understanding of the required tests and the possible caveats associated with each test to properly interpret the results.

Approach to Laboratory Analysis of Concrete

When MRD is suspected of playing a role in the premature deterioration of a concrete pavement, laboratory tests are essential to help understand the underlying mechanisms at work. Optical microscopy using the stereo microscope and the petrographic microscope are the most versatile and widely applied tools for diagnosing causes of MRD. Electron microscopy is becoming more prevalent, especially for chemical identification of reaction products and other secondary phases using energy dispersive spectroscopy. Analytical chemistry plays a very important role in determining key parameters of the concrete (e.g., w/c , chloride content). XRD is applied in some cases but is not widely used in the analysis of deteriorated concrete.

Often, when diagnosing concrete distress, there is no clear answer as to which single distress mechanism caused the failure. This has been referred to as "the straw that broke the camel's back" theory (Erlin 1993) where multiple distress mechanisms are active and it is the combination of these, in concert, that lead to the failure of the concrete. Various types of distress mechanisms can occur simultaneously in concrete and each can incrementally contribute to the ultimate failure of the material. Although in many cases it is difficult to attribute concrete distress to one mechanism, in a majority of cases, the MRD can be easily diagnosed. This is often accomplished by use of the optical microscope alone. As a result, most laboratory diagnostic procedures have focused heavily on the use of optical microscopic methods. Analytical chemistry methods have also been used, such as staining techniques and determination of parameters such as w/c . Other techniques, such as SEM and XRD, have also been widely used by researchers and are becoming more common for forensic investigations of concrete failures.

When initiating the study of deteriorated concrete, or any material, an analysis plan of how to approach the problem must be followed. This plan very often reflects a process of elimination; rather than proving what the problem is, prove instead what it is not. In this way, diagnosis of the MRD responsible is achieved. The basic flow of a typical laboratory analysis is presented in figure 1-6. The general approach is to start very broadly, inspecting the concrete by eye. As the concrete is examined the analyst should look for diagnostic features, which are essentially a condition or physical property of the concrete that will assist in diagnosis. After evaluating or assessing the core visually, a hand lens or stereo microscope can be used to look more closely at interesting features. In some cases, as a result of this visual and stereo microscope analysis, the probable or certain cause of distress is identified. In most cases, a few potential MRD types can be eliminated and further analysis can then focus on those remaining. At this point, the analyst must decide which examination technique can be performed to confirm a given MRD, or eliminate other MRD types, thereby narrowing the choices to the most probable mechanism.

A summary of the recommended process is as follows. A sample of concrete exhibiting distress comes into the lab and is first visually inspected. Upon completion of the visual inspection, specimens are produced from the core sample and observed using the stereo OM for initial optical analysis. It is common at this stage to employ staining techniques to help identify ASR or sulfate phases. Next, the specimen is viewed in the petrographic microscope and/or SEM, as required. This process of using the stereo OM, petrographic OM, and SEM is iterative and it is quite common to view the same specimen in all three instruments. Staining in particular can assist in the optical evaluation although it may interfere with SEM analysis.

Although a trained petrographer can use the petrographic microscope and identify practically all minerals and aggregate reaction products present in a concrete specimen, this often requires thin section

preparation or detailed analysis of picked grains with refractive index liquids. This requires a highly skilled concrete petrographer to analyze and interpret the complex and vast array of information revealed by the petrographic OM. In contrast, the SEM is an instrument almost any laboratory technician can learn to operate. Another advantage of the SEM is the simple presentation of the results in a form engineers, technologists, and scientists can all understand. Both x-ray mapping and x-ray microanalysis are very useful ways of identifying components of the microstructure. However, the use of the SEM has some disadvantages.

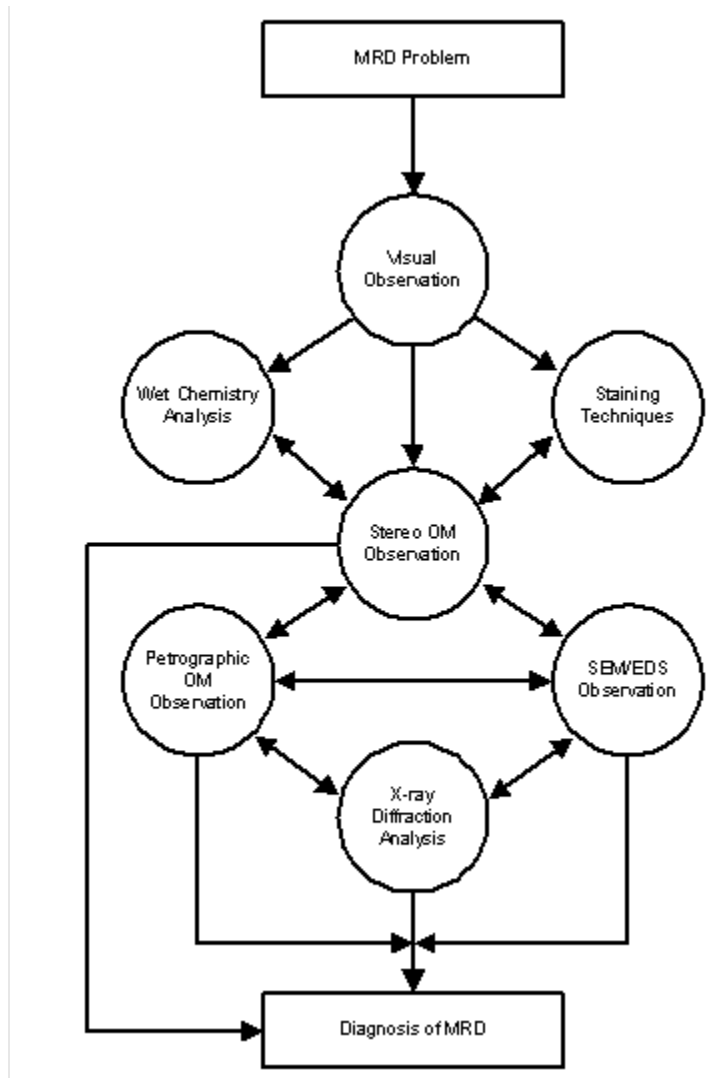


Figure 1-6. Fundamental process for analyzing a concrete MRD sample.

Cracking problems in the CSEM, microanalysis problems in the ESEM, and a much higher initial cost with a significant on-going maintenance cost associated with its operation.

When analyzing a concrete specimen, the concrete should be viewed as an entity consisting of a system of four principal components: air, hydrated cement paste, coarse aggregate, and fine aggregate. All available methods to examine the system and its components should be used, looking for all features that will help in the diagnosis. The ability to establish certain features as being normal greatly helps in deducing the cause of the problem. For example, no apparent coarse aggregate cracking all but eliminates aggregate freeze-thaw deterioration as a cause of distress. As another example, the presence of an adequate, un-compromised air-void system helps rule out paste freeze-thaw damage as the primary distress mechanism. It is emphasized that a systematic examination of all components of the concrete is crucial to determining the cause of failure.

In the end, the proper examination of concrete requires the application of independent, unbiased testing methods in a uniform and controlled approach. The number of required tests is determined by the complexity of the MRD while the implementation may depend upon various factors including scheduling of laboratory equipment and personnel.

Procedures and Data Collection for the Analysis of Deteriorated Concrete Pavement

The purpose of this section is to provide an overview of recommended analytical procedures used to examine MRD in concrete pavements. To help facilitate this standard analysis, data sheet templates were designed to follow a core through the laboratory evaluation. These are provided in the guideline presented in Volume 2 of this Final Report. The use of these data sheets assists in the laboratory analyses and data interpretation in a number of ways. First, these data sheets provide a framework for the “questions asked about the material” and are intended to be consistent with the requirements of ASTM C- 856 *Practice for Petrographic Examination of Hardened Concrete*. It is noted that the individual questions and data queried in these forms do not constitute the full implementation of ASTM C- 856. Instead they focus on the analysis of deteriorated pavement concrete, which is only a small part of the scope included in this very broadly applied Practice.

Another benefit of using data forms is that they serve as the “lab notebook” for recording and archiving laboratory results. As an example, careful inspection with a stereo microscope alone can often reveal the reason for concrete failure. Although a diagnosis of MRD based on stereo microscope observations may be correct, it is often necessary to use other techniques to confirm the initial diagnosis. The data sheets provide a systematic way of gathering and archiving the results of multiple laboratory analyses that may be conducted by multiple technicians, often in different laboratories. Finally, personnel outside the laboratory want to know that all possible deterioration mechanisms were considered and that no rush to judgment was made on limited data. By using standard data collection forms, the diagnostic indicators laboratory personnel were looking for will be clear to others. This reporting protocol helps provide an understanding of the decisions made by laboratory personnel to arrive at a diagnosis of material failure. The data sheets provided in the guideline are broken up into the following major categories roughly corresponding to the steps outlined in figure 1-6:

- Sample Receipt and Cataloging
- Visual Inspection
- Stereo Optical Microscope Examination
- Staining Tests
- Petrographic Optical Microscope Examination
- Scanning Electron Microscope Examination
- Chemical Tests
- X-ray Diffraction Tests

One copy of the completed laboratory data sheets, along with sampling and field data sheets, should accompany each core through the laboratory evaluation. Within each data sheet, tests and diagnostic features are presented in a table format along with possible or common results. It is up to the analyst to observe and make judgments about the concrete relative to the questions posed on each data sheet. Laboratory personnel should complete only those data sheets that apply to analyses conducted. These observations are combined with all other data to arrive at a diagnosis of the MRD observed. Details regarding the data collection process are provided in the guideline in Volume 2 of this final report.

Summary on Recommended Laboratory Testing Procedures

The diagnosis of MRD in PCC often requires the use of various laboratory procedures for identifying the extent and mechanism of distress. These laboratory tests do not always lead to absolute characterization of the distress for a variety of reasons. First of all, most cases of concrete distress occur as the result of multiple distress mechanisms. As such, it is often difficult to isolate the specific cause of failure or even determine the principal cause of failure. Second, the laboratory methods commonly used often provide results that may be interpreted differently, depending upon the motivation or objectivity of the analyst. To minimize the latter case of misinterpretation, it is advised that a thorough, complete examination of the concrete be performed using the data collection forms and approach described.

Clearly, the most useful tools for examining concrete are the stereo OM and the petrographic OM and/or the SEM. There is a significant body of technical information available discussing OM observations of concrete and concrete distress. This information is useful as a benchmark when evaluating a specific concrete specimen. The SEM is quickly becoming an equally valuable tool for evaluating concrete. It is advised that anyone charged with the examination of MRD in concrete become familiar with this equipment and the capabilities it offers.

Finally, the general approach of "asking the materials questions" must be followed. As is discussed in the next section, it is often the case that a process of elimination is required to determine what distress is not present, thereby leading to a shorter list of possible distress mechanisms. Also, it is important to remember that the concrete as observed may have undergone a significant metamorphosis over its service life and the degradation seen may be the final product of years of exposure. It is only through the careful and methodical application of the described laboratory methods that the true cause of distress may be identified.

Background to Data Analysis and Interpretation of MRD in Concrete Pavements

In order to make an accurate diagnosis of MRD, four primary sources of data must be analyzed. The first source of data is that obtained through a review of existing construction records that details the materials used, mix design, and conditions during construction. Unfortunately, construction records are not always available and when available are not always of sufficient accuracy to assist in the diagnosis. Next, site-specific information not directly related to the construction of the project under investigation can also contribute valuable insight. Specifically, climatic information can be used to assess the environment to which the project was subjected and soil analysis can be used to investigate the possibility of whether some potentially harmful compounds (e.g., soluble sulfates) were naturally present. A third source of valuable records are the responsible agency's maintenance records, which will not only reflect the level of maintenance activities applied, but also the amount and type of deicers that may have been used. The last and most important source of data is from laboratory analyses, which are almost always required to reach a conclusion regarding the cause of the observed distress.

Unfortunately, there are no definitive sources of information on how to diagnose a concrete failure given the wide range of material applications, environments, mix designs, and possible reasons for failure. To develop a protocol for diagnosing MRD in PCC pavements, it is necessary to assemble known information from numerous sources. Then, the underlying relationships between various observed diagnostic features must be understood, again requiring reference to many sources of information. Further, there is currently no available protocol for identifying MRD in PCC pavements. Without a defined method, agency personnel are stymied in their quest to determine the deterioration mechanisms that are shortening the life of PCC pavements. In the process of developing the protocol presented in the guidelines, the following summary of diagnostic features for each common MRD was gleaned from the literature.

Diagnostic Features for Paste Freeze-Thaw Deterioration

When moist concrete is exposed to alternating cycles of freezing and thawing, internal deterioration can result in the paste, as described previously. Paste freeze-thaw deterioration is observed on the pavement surface as scaling and/or map cracking, typically appearing first near joints and free edges. Deterioration may have progressed to the point where the surface is pitted as the paste disintegrates, exposing coarse aggregate particles, which eventually may work free. Confirmation of the diagnosis requires laboratory examination, primarily through optical microscopy.

Concrete affected by paste freeze-thaw damage can be identified through the presence of microcracking in the paste using the petrographic optical microscope. If the freeze-thaw cycling has led to scaling, these microcracks will predominantly run parallel to the wearing surface. The best method to determine if paste freeze-thaw is the primary distress mechanism is a microscopic characterization of the air-void system, following ASTM C 457 procedures to assess the adequacy of the entrained air system (Walker 1992). Analytical systems to perform ASTM C 457 are not common, but are available. Often, it is easier for the analyst to purchase the necessary hardware and write the necessary controlling software. For this research, the latter approach was adopted (Sutter 1998).

On normal strength concrete (relatively high w/c), an entrained air-void system producing a Powers spacing factor of 0.20 mm or less will typically provide good freeze-thaw protection. This value was empirically established primarily through laboratory freeze-thaw testing using ASTM C 666 (AASHTO T 161). While the Powers spacing factor is not considered to be a truly definitive measure of freeze-thaw protection, it is still used as the standard method of quantifying the distribution of entrained air in concrete. Other measures for characterizing entrained air have been proposed (Walker 1980; Philleo 1983; Attiogbe et al. 1993), but none have been adopted for general use.

The minimum requirements for an entrained air system in higher strength concrete (lower w/c) are less clear. Some laboratory research has indicated that, as the w/c decreases below 0.5, the spacing factor required for freeze-thaw durability increases. While accepted recommendations for a larger spacing factor for higher strength concrete have not been published, ACI 318 - 95 (Section 4.2.1) permits a 1 percent reduction in the recommended air content for concrete with specified compressive strengths above 34.5 MPa.

The microscopic examination of the air-void system in hardened concrete can take the form of a point count, linear traverse, or an areal traverse. All three methods rely on measurements obtained from a polished plane surface of concrete. The general mathematical methods used to extrapolate measurements obtained from a two-dimensional surface to three-dimensional space are known as stereology (Russ, 1986). The mathematical derivations for volume fraction relationships are based upon measurements in a two-dimensional slice of the volume, as is the case with a polished microscope specimen. The fundamental relationships leading to volume fraction estimation depend upon equivalence between the volume density of quantities measured on a plane section, including point, line, or area fractions. The fundamental relationships are shown below:

$$P_P = L_L = A_A = V_V \quad (1-3)$$

where:

P_P = Fraction of total points counted falling in phase of interest.

L_L = Fraction of total line length traversed falling in phase of interest.

A_A = Area fraction of phase of interest.

V_v = Volume fraction of phase of interest.

Both methods of volume fraction estimation, point counting and linear traverse, have been applied to air void measurement in concrete.

In addition to the total volume of air in hardened concrete, ASTM C 457 details procedures for determining a Powers spacing factor. Powers (1949) developed two expressions for a spacing factor, both of which require a determination of the total air-void specific surface. It can be demonstrated that the total volume of air voids and their total specific surface can be estimated from the mean air-void intercept or chord length obtained from a linear traverse. Assuming all air voids to be spherical and using geometric probability concepts, the total specific surface, α , expressed in terms of the average chord length, $\bar{l}$, was shown to be:

$$\alpha = 4 / \bar{l} \quad (1-4)$$

Powers first spacing factor expression was obtained by simply calculating the volume of cement paste per unit area of air-void surface. This is given as:

$$\bar{L} = p / \alpha A \quad (1-5)$$

where:

$\bar{L}$ = Spacing factor, in units of length.

α = Total specific surface of the air voids, in consistent units of length⁻¹.

p = Paste content, in volume percent of concrete.

A = Total volume of air voids, in volume percent of concrete.

The second spacing factor expression is based on a hypothetical system of equal sized spherical voids uniformly distributed throughout the paste phase. The size of each of these hypothetical voids is determined by setting their specific surface ($3/R$) equal to the total measured specific surface of the true void system and then solving for the resulting sphere radius, R . By making the total air content of the hypothetical system of voids equal to the measured value of air content, the number of hypothetical voids is then determined.

The cubic packing of this hypothetical void system can be visualized as a system of equal sized adjoining cubes of cement paste with an internal air void located at the center of each. The maximum distance from anywhere in such a system to the nearest surface of an air void is the distance along the cross diagonal from a cube corner to the enclosed void surface. This distance is equal to one-half the length of the diagonal minus one-half the sphere diameter. The Powers spacing factor thus obtained is:

$$\bar{L} = 3/\alpha [1.4(1 + p/A)^{1/3} - 1] \quad (1-6)$$

Powers recognized that neither expression for spacing factor provides a true measure of void spacing. Assuming that both expressions overestimate the true average void spacing, he recommended using the smaller spacing factor obtained from the two equations. Equation 1-5 yields a smaller factor for p/A less than 4.33, and Equation 1-6 gives the smaller value when p/A is greater than 4.33.

It has generally been found that if the spacing factor is adequate, then the cement paste should be protected against freeze-thaw damage. But it is readily acknowledged that paste freeze-thaw damage can occur even in concrete having what appears to be an acceptable air-void system. This type of damage is commonly associated with scaling of the surface. Microscopic observation of the microstructure may reveal a system of cracks running parallel to the wearing surface, with crack spacing typically widening as the distance from the exposed surface increases (Walker 1992). This type of manifestation is commonly associated with poor finishing in which either bleed water was trapped or the surface was overworked, compromising the air-void system. The application of deicers can also contribute to this cracking by accentuating the stresses caused by freezing and thawing at the surface. Similar cracking has been reported at joints, except that the cracks may run parallel to the joint face as well.

Paste freeze-thaw deterioration is a physical process, and therefore does not directly involve deleterious chemical reactions. In light of this fact, one would expect that the observation of microcracking in the paste, not apparently associated with a chemical process, might thus be due to a physical process such as freezing and thawing. Unfortunately, this is not always the case as such microcracking could be the result of aggregate dilation. In damaged concrete it is very common to find secondary deposits (e.g., ettringite, calcium hydroxide, etc.) in air voids and/or cracks. It has been stated by some that these secondary deposits in air voids might compromise the freeze-thaw durability of the paste resulting in damage (Niemann and Lehtonen 1997; Ouyang and Lane 1997). Others have stated that the infilling of voids and the secondary deposition of compounds such as ettringite are not the cause, but instead the result of damage that has already occurred due to other deterioration mechanisms. It is common to measure the air-void system parameters to determine if they are adequate and to suspect paste freeze-thaw damage as at least a contributing factor if they are not.

Diagnostic Features for Deicer Scaling/Deterioration

Distress related to deicer use is most often associated with surface scaling and delamination, which is almost unique to this distress type. The deterioration is observed in the paste, exposing the coarse aggregate particles. This is in contrast to a distress such as "popouts" that is characterized by fractured aggregate particles surrounded by intact paste. If the concrete under evaluation is subjected to deicer applications and is showing signs of scaling, deicers are likely contributing to the distress.

Chemical test methods, such as AASHTO T 260, can be used to measure the chloride ion concentration in relation to depth of a concrete specimen. The presence of a high chloride ion concentration at or near the scaling surface is an indicator that the distress is likely related to deicer applications.

Determination of deicer deterioration at joint and crack locations is a much more difficult task. Rarely is the deicer directly responsible for deterioration. It contributes by increasing the level of saturation, increasing the solubility of some compounds, and potentially increasing the alkalinity of the pore. Thermal shock may also contribute to the occurrence of distress. Under these circumstances, the concrete becomes more susceptible to other distress mechanisms such as paste and aggregate freeze-thaw deterioration as well as ASR. The deicer may also provide a source of soluble sulfates, resulting in external sulfate attack.

Although the primary distress mechanism may obscure the contribution of the deicer to the deterioration, there are some diagnostic features that are indicators of whether deicer deterioration is at least a contributor to the observed distress. Muethel (1997) examined concrete at crack locations using phenolphthalein to identify calcium hydroxide depletion and permeability measures to estimate coarsening of the paste. He was able to identify zones at the concrete/crack interface that have significantly altered microstructure compared to the mass concrete, speculating that these differences in microstructure have resulted in joint/crack deterioration and staining. He further states that primary leaching agent is carbonic acid, which aggressively attacks concrete in a high salinity environment. The

presence of chemical deicers thus accelerates the leaching process. Marchand et al. (1994) described a similar process.

It is logical to assume that salts would concentrate in joints or cracks due to infiltrating melt water produced from the deicer application. If the chloride ion concentration were mapped from the joint interface into the concrete mass, the gradient would likely be one in which the concentration would be highest at some distance inward from the joint. The concentration at the interface would decrease as the flow of water in or through the crack/joint would have a tendency to wash the deicers away while wetting and drying cycles would concentrate them just below the interface surface. Thus, chemical methods could be employed to determine chloride ion concentrations at various distances from the joint/crack face to measure this phenomenon.

Polished concrete slabs can be examined by noting to what depth the cracking is present. Commonly, a pavement undergoing deicer scaling will have microcracking isolated in and parallel to the surface. This is very similar to common paste freeze-thaw deterioration (Walker 1992), and in fact the mechanisms are closely related. In addition, if the air-void system parameters in the lower portion of the specimen are adequate, and the paste near the finished surface is devoid of air, poor finishing or curing is likely the responsible mechanism.

A marginal air-void system throughout the concrete mass can initially manifest itself as deicer scaling, but the distress is likely to be progressive in nature as the problem will not be isolated to a weak surface layer. This would be best categorized as paste freeze-thaw deterioration.

Petrography can also be employed to assist in the diagnosis of deicer deterioration. Carefully prepared polished thin sections could be used to identify salt crystals that might be present. Also, changes in the paste porosity/texture will be identifiable if care is taken in sample preparation. Some researchers are convinced that common deicers react with cement paste to form chloroaluminates and oxychlorides, compounds that can be observed through optical petrography and electron microscopy.

Deicer distress can appear outwardly similar to ASR and aggregate freeze-thaw deterioration in some manifestations. Wolter speculates that ettringite, in its "amorphous" form, has been confused with alkali-silica gel. An SEM with EDS capability can identify the minerals definitively. In the case of deicer distress, ettringite filling the interstitial pore space can be observed using an SEM (Wolter 1997). The SEM preferably should be an LVSEM or ESEM to reduce the drying shrinkage cracking caused by a high vacuum in the CSEM. Wolter (1997) gives the following characteristics of deicer distress that can be identified using optical microscopy:

- Microcracking propagating around aggregate particles and through the paste only.
- Aggregates relatively intact with virtually no reaction rims of silica gel observed,
- Deterioration within the paste only.
- Microcracking within the paste occurring predominantly sub-parallel to the deteriorating surface.
- Secondary deposits of ettringite within the air-void system and microcracks.
- Less microcracking and secondary deposits as distance increases from the deteriorating surface.

When diagnosing possible deicer distress, the orientation of the sample is important. Ideally, the sample should contain the vertical joint surface, so that the conditions with distance from the joint can be noted. Carbonation, microcracking, and the quantity of ettringite often change dramatically over distances of 25 to 50 mm, and samples with unknown orientation to the joint are virtually useless (Wolter 1997).

Thin section analysis should be used to detect ettringite growth within a functional air void. Wolter warns that using melted carnauba wax prior to polishing may alter ettringite due to the heat involved. Therefore, as much analysis as possible should be done before the wax is used.

It has been shown that concrete subjected to repeated applications of deicers and wetting and drying cycles develops a coarser pore structure, characterized by larger, interconnected pores. Concrete permeability thus increases, making it more susceptible to many other distresses. This connection has been described by a number of researchers. For example, Basheer (1994) has proposed a single deterioration of concrete-permeability interaction model that links concrete permeability to corrosion of steel, frost damage, chloride ingress, carbonation, sulfate attack, salt attack, alkali attack, acid attack, alkali-aggregate reactions, and abrasion. It therefore may be possible to use the increased permeability associated with deicer deterioration as a diagnostic tool by comparing concrete obtained near joint and crack interfaces to that obtained from a slab interior.

Two widely accepted standard test methods for measuring concrete permeability are the British test method (British Initial Surface Absorption Test) that is not commonly used in the United States and AASHTO T277 (ASTM C 1202) rapid chloride permeability test (RCPT). Many other test methods exist, but none has gained greater acceptance than the RCPT. SHRP researchers recommended modification to the AASHTO T 277 test to make it more rapid (Zia et al. 1993). It was also advocated by SHRP researchers that a "pulse pressure" method for concrete permeability be adopted to measure water permeability in the laboratory (Roy et al 1993a; 1993b). A third SHRP test method was proposed for field evaluations in which the rate of airflow through a concrete surface under an applied vacuum is measured to estimate permeability. Unfortunately, this test is not considered sufficiently quantitative for prediction of actual permeability and thus results should only be used as an indicator of permeability (Whiting et al. 1994). More recently, a concrete sorptivity test has been under development and is under review by ASTM that may provide an indirect method to assess concrete permeability.

Mobasher and Mitchell (1988) published the results of a large study investigating the applicability of the RCPT. They found that the "test is valid and can be used with confidence." They investigated the repeatability of the test, finding the single operator coefficient of variation (COV) of a single test result to be 12.3 percent and the multi-laboratory COV to be 18.0 percent. Misra et al. (1994) report that the RCPT can be used as an important tool for quality control, inspection, and design, although further research is needed to examine the effect that pore solution chemistry has on the results. This is a very important finding from the perspective of using this test as a diagnostic tool, as the pore solution in concrete near joints and cracks will be different than that elsewhere due to the ingress of deicing chemicals.

Feldman et al. (1994) reported in their investigation of the RCPT how changes in experimental conditions and specimens affected results. It was observed that the test affected the pore structure and resistivity of concrete, particularly relatively young concrete. It was also stated that simple measurement of initial current or resistivity gave the same ranking for the four concrete specimens tested, and therefore this might be able to replace the RCPT. It is noted that all the concrete produced had a w/c of 0.55, and that their findings need to be verified for lower permeability concrete, and that made with blended cements.

Although the RCPT has been embraced by many SHAs due to its ease of use, it suffers some limitations that make it impractical when evaluating some mixtures. The three main limitations are: 1) the current passed is related to all ions in the pore solution and not just chloride ions, 2) the measurements are made before a steady-state migration is achieved, and 3) the temperature of the specimen increases due to the applied voltage (Stanish et al. 2000). The first limitation is most problematic for the assessment of concrete permeability in mixtures containing various admixtures (e.g., accelerators, corrosion inhibitors, etc.) that will affect the ion concentration of the pore solution.

Diagnostic Features for Aggregate Freeze-Thaw Deterioration

Field identification of aggregate freeze-thaw deterioration usually starts with a visual inspection of the pavement surface. Because of the increased access to moisture, the intersections of transverse and longitudinal joints and pavement edges are locations where the characteristic fine-line cracking pattern first appears. The surface staining that frequently precedes this cracking can also serve as an indication

of aggregate freeze-thaw deterioration below the surface. While the characteristic cracking and staining are strong indicators of a problem, in and of themselves they are not proof positive that aggregate freeze-thaw deterioration is the mechanism at work. The common practice of associating aggregate freeze-thaw deterioration with observations of "D-cracking" should thus be avoided.

For positive diagnosis, cores must be obtained either directly through or in the vicinity of distressed joints or cracks to confirm the source or the surface deterioration. For concrete that is exhibiting surface cracking associated with aggregate related freeze-thaw, it is common for the bottom of the core to be extensively damaged. In situations where the surface evidence is staining rather than characteristic cracking, observations of microcracking of some coarse aggregate particles near the bottom of a core can confirm early stages of aggregate freeze-thaw deterioration. Finally, it must be established that the aggregate deterioration is a physical process and not related to aggregate reactivity.

In a comprehensive study of procedures for identifying frost-susceptible aggregate particles, Larson and Cady (1969) reported observing four failure modes. These included aggregate shattering, bed-plane disintegration, aggregate paste bond failure, and general disintegration of the paste matrix. They also reported that even though the aggregates used in their study were selectively fractionated by size, specific gravity, and mineralogy into relatively homogeneous samples, a relatively small number (5 to 10 percent in some cases) of individual aggregate particles were typically responsible for the disruptive behavior.

It is best to diagnose aggregate freeze-thaw deterioration through optical microscope analysis of prepared concrete specimens. Analysis can be conducted on both polished slabs and thin sections. Aggregate-paste bond failure through the interfacial zone and possibly disintegration of the paste matrix resulting from the expulsion of water from the aggregate during freezing are diagnostic features of aggregate freeze-thaw deterioration. Diagnosis based on these features is less certain as the aggregate particle remains intact and thus it can be easily confused with paste freeze-thaw deterioration. The analyst needs to examine the air-void system in undamaged paste and the pore structure of the aggregate to determine the most likely distress mechanism.

As discussed previously, it is well established that the aggregate pore system is the most important factor in determining its freeze-thaw durability. In an effort to classify aggregates according to their potential freeze-thaw durability, Verbeck and Langren (1960) developed three general aggregate groupings based on their pore systems. In the first group are aggregates with a low enough porosity and/or permeability and a high enough strength so that they can accommodate the internal changes due to freezing by elastic action. Aggregates in this first group are considered freeze-thaw durable. In the second group are aggregates that have an internal pore system small enough to compete successfully with the cement paste for moisture and therefore tend to become critically saturated. At some critical maximum size these aggregates will generate excessive hydraulic pressures that cannot be accommodated by elastic action and therefore tend to not be freeze-thaw durable. The third group includes aggregates with a high enough permeability along with adequate mechanical properties so that hydraulic pressures from freeze-thaw events are accommodated by elastic action. While aggregates in this third group remain sound during repeated freezing and thawing, they may cause distress to the surrounding paste system or interfacial zone because of the volume of water expelled.

While useful as a means of describing different behavior, the non-uniform nature of many aggregate sources makes this grouping impractical for classifying aggregate sources. But through classification of the aggregate mineralogy and pore structure, a skilled petrographer can attempt to diagnose whether a given aggregate is to blame for an observed physical deterioration of a concrete pavement. This in combination with the observed cracking and/or deterioration should be sufficient to distinguish the physical mechanism at work.

Diagnostic Features for Alkali–Silica Reactivity (ASR)

ACI (1998), Farny and Kosmatka (1997), ACPA (1995), and Walker (1992) have compiled lists of diagnostic features characteristic of concrete pavements affected by ASR. Visual signs of ASR include map cracking on pavement surfaces, cracking that is more open on the surface, dark or light reaction rims around the coarse aggregates, and visible cracks radiating from within aggregates. A whitish powder, which is calcium carbonate leaching from the deteriorated concrete, in the specimen or visible on the pavement surface, is another strong indicator that ASR is occurring.

The Cement and Concrete Association of Australia (CCAA)(1996) prepared guidelines to properly determine if an existing concrete structure is in fact undergoing alkali–silica reactivity. Visual clues include cracking and expansion of the structure such as closing of joints and displacement of fixed members. Other signs that must be noted include damp patches and efflorescence. Walker (1992) considers the following five items for diagnosis of ASR:

- Crack pattern.
- Structural evidence of expansion.
- Verification of the presence of reactive silica in the aggregate.
- Exudations, coatings, and pore fillings.
- Laboratory investigation of sampled concrete.

Walker (1992) presented a method for distinguishing between plastic and drying shrinkage, stating that the crack pattern commonly associated with ASR affected concrete is similar to that generated when mud dries and shrinks, or plastic shrinkage cracking. This generates an irregular honeycomb pattern in unrestrained concrete as each portion of the surface pulls away from adjacent portions. In reality, it is not a surface shrinkage phenomenon, but instead an expansion of the underlying concrete that produces the crack pattern. The differential expansion is the result of the higher moisture content of the substrate concrete that allows the gel to swell.

The actual pattern of the cracking depends on the constraint to which the concrete is subjected. If the structure is unrestrained, it will be in a honeycomb pattern. But reinforcing steel in concrete structures, which are considerably longer than they are wide (such as pavements), will not allow the concrete to expand equally in all directions. In these cases, initial manifestations of surface cracking will occur perpendicular to the direction of expansion. In an affected pavement, this cracking will thus run longitudinally parallel to the pavement edge. At the same time, restraint will slow down the deleterious nature of ASR in reinforced structures (Walker 1992).

In most advanced cases of ASR, the silica gel will be visible on the concrete surface (Walker 1992). The gel is most commonly dehydrated and carbonated, being an opaque white or efflorescent, and under traffic may polish to a translucent glaze. Non-dehydrated gel is translucent and is moist or sticky.

Stark (1991) advocated a staining method for determining whether ASR gel is present based on the reaction of the ASR gel and uranyl acetate. ASR gel treated with a uranyl acetate solution fluoresces under UV light, making it possible to detect ASR gel before it is visible to the naked eye. There are some difficulties in effectively using this test, however. One problem is that, although it is generally accepted that treatment with uranyl acetate will cause the ASR gel to fluoresce, it has also been demonstrated that other non-deleterious compounds can also fluoresce. This leads to false positives, resulting in the necessity for additional analysis if a positive result is obtained. A second concern surrounds the use of uranyl acetate, which is a radioactive compound. The fact that uranyl acetate is a hazardous material means that special precautions regarding handling the compound and specimens treated with it are required. Thus, there is a reluctance to employ this method.

More recently, a new test procedure has been developed for the detection of ASR reaction products (Civil Engineering News 1997; Engineering News Record 1997; Guthrie and Carey 1997). This method uses

sodium cobaltinitrite and rhodamin B solutions to stain the ASR reaction product. After treatment with the solutions, the specimens can be visually assessed with a hand lens or using an optical microscope. Regions where ASR gel is present stain either yellow or pink. It is observed that yellow staining is associated with a massive precipitate having a distinctive gel-like morphology and granular precipitate that appears to consist of crystals that have grown from the gel (Guthrie and Carey 1997). It was determined that the yellow staining regions are potassium, calcium, silica, and sodium gels resulting from ASR. Because this test is relatively new, little field experience currently exists regarding its applicability and reliability.

Guthrie and Carey (1997) compared the results of this new staining method to the uranyl acetate method, finding excellent agreement between the two methods with the added benefit that the dual staining technique provided additional information through the appearance of both yellow and pink staining (the uranyl acetate test tagged both). The major benefit of the cobaltinitrite/rhodamin B method is that the chemicals are not radioactive and UV light is not required for illumination, thus conventional optical microscopes can be used in the analysis.

Farny and Kosmatka (1997) identify several optical methods for use in determining ASR. It is recommended that a close visual examination should be conducted to identify signs of distress, such as microcracking and separation of aggregates from the paste. Some type of magnification, whether through a strong magnifying glass or microscope, is needed for this examination. To positively attribute the distress to ASR requires confirmation of the presence of an ASR reaction product. A petrographic analysis or analysis using a scanning electron microscope can positively identify whether an alkali-silica gel is present (Thaulow et al. 1996b).

Positive identification of ASR also requires that some of the aggregate within the paste be recognized as reactive or potentially reactive and that these aggregates must show signs of reaction. Aggregate exhibiting internal fracturing that extends to the concrete matrix is a common diagnostic feature in concrete affected by ASR. If only the fine aggregate is reacting, cracking in the matrix may be found that does not affect the coarse aggregates. A network of internal cracks connecting reacted aggregate particles is a strong indication that ASR has occurred. It has also been observed that silica gel may appear as dark areas or rims around the edge of reacting aggregate.

Walker (1992) provides a detailed description of microscopic diagnostic features observed in ASR affected concrete. Aggregate will be altered by the reaction, as evidenced by one or more of the following features:

- Rims on the aggregate that seem to make the aggregate exterior less permeable.
- Gel-filled cracks extending into the paste.
- Hollowed-out centers of sedimentary rocks resulting from the consumption of the original cement between sand grains by the reaction.
- Aggregate particles that have been cracked by gel.

Exuding liquid gels will saturate the paste, fill cracks and voids, and will ooze onto the pavement surface. These gel deposits become more common with depth. The interior of the concrete will have large cracks that may not be filled with gel because they are the result of expansion elsewhere.

Thaulow and Jakobsen (1997) list the following four main microscopic features that can be used to diagnose ASR:

- The presence of alkali-silica reactive aggregate.
- A characteristic microcracking pattern.
- The presence of alkali-silica gel in cracks and voids.
- Calcium hydroxide (CH) depleted paste.

If the number of reactive aggregate particles is sufficiently high, a continuous crack pattern will develop in the concrete. The cracks will follow a path of least resistance, running through the cement paste and reacting aggregates and, in rare instances, even through sound aggregate particles (Thaulow and Jakobsen 1997).

Some unique features of the alkali–silica gel discussed by Thaulow and Jakobsen include the fact that it is normally a colorless, isotropic material having a low refractive index of 1.46 to 1.53, but on occasion it may be partly crystalline, having an orange interference color in crossed polarized light. The gel may also be observed replacing the outer portion of the aggregate particle. Ettringite commonly is present in ASR affected concrete, and is easily distinguished by the slight birefringence of the ettringite formation. Thaulow and Jakobsen (1997) suggest using fluorescent epoxy impregnation of thin sections to assist in making the distinction between the ASR gel and ettringite. They also mention that in intensive ASR, the CH can be dissolved, "leaving a black and opaline shining paste" when observed in cross-polarized light. This dark paste area is commonly confined to narrow zones around reactive aggregates and along gel containing cracks.

In another paper, Thaulow et al (1996b) discuss using SEM equipped with an EDS to investigate the composition of alkali–silica gel in deteriorated concrete. In the study described, both optical and electron microscopy were used to view the same areas of prepared, epoxy impregnated thin sections. The SEM/EDS analysis confirmed the findings of the optical microscope, positively identifying ASR gel in all locations identified by the optical microscope. Furthermore, the SEM/EDS was able to locate ASR gel in some cracks in which the optical microscope lacked sufficient magnification to make a positive identification. The SEM/EDS was also able to determine the composition of the observed gel, something that the optical microscope could not do. An interesting finding of the compositional analysis is that, in most cases, the gel in air voids was richer in calcium compared to gel in cracks in the aggregate, and that the calcium content is directly related to the proximity of the gel to the cement paste.

It is noted in ACI 221.1R (ACI 1998) that when observed in thin section, disseminated calcium hydroxide is depleted in the cement paste in the vicinity of reactive-aggregate particles. This observation is often noted before gel formation and cracking and is therefore useful in detecting early signs of ASR.

Diagnostic Features for Alkali–Carbonate Reactivity

The visual identification of ACR as the mechanism causing damage within concrete can be difficult because the manifestation of ACR mimics those of various other MRDs. The visual signs common to all AAR are cracking, closing of joints, movement, displacement of members, and pop-outs. In unreinforced concrete, map cracking is the most common form of cracking. Reinforced concrete structures exhibit cracks that run parallel to the reinforcement (CCAA 1996). Because these visual signs are common to both ASR and ACR, it is very difficult to distinguish one from the other through unaided visual assessment. Ozol (1994) determined that in ACR affected concrete, there is no distinct feature or geometry of cracking that precisely identifies the cause to be ACR. However, there is one major difference: an ASR product gel always accompanies ASR whereas a gel never accompanies ACR (ACI 1998).

It is recommended by Farny and Kosmatka (1997) that both the concrete and the carbonate rocks be examined petrographically in accordance with ASTM procedures C 856 and C 295, respectively, if ACR is suspected as the deleterious mechanism. As discussed in Appendix A, ACR susceptible aggregate has unique features that can be used in the process of diagnosis. Swamy (1994) identified three aggregates with which carbonate reactions commonly occur. These aggregates are calcitic limestones, dolomitic limestones, and fine-grained dolomitic limestone aggregates containing interstitial calcite and clay. Farny and Kosmatka (1997) outline aggregates that have potential for ACR in the following manner:

- Aggregates with a clay content, or insoluble residue content, in the range of 5 to 25 percent.
- Aggregates with a calcite to dolomite ratio of 1:1.

- Aggregates with increasing dolomite volume up to a point at which interlocking texture becomes a restraining factor.
- Aggregates with discrete dolomite crystals (rhombs) suspended in a clay matrix.

Gress (1997) and Ozol (1994) also report that the observation of large dark gray dolomitic limestone coarse aggregates having cracks radiating into the concrete paste is one indicator of ACR. Walker (1992) also made this observation. Another characteristic of ACR mentioned by Gress (1997) was the distinct reaction rims formed around the reactive aggregates, although Ozol (1994) pointed out that rim zones may or may not be produced on reactive particles, and therefore are not in and of themselves definitive in diagnosing ACR.

Walker (1992) states that the "signs indicating alkali-carbonate reactivity can be very subtle because the reaction does not cause the growth or exudation of any characteristic reaction products, such as the gel associated with alkali-silica reactions." She argues that the best diagnostic approach is through petrographic examination of the aggregate to identify microstructure and degradation. Key aggregate microstructural features are the "small rhombic crystals of dolomite suspended in a calcite, micrite matrix that contains the fine, particulate, insoluble constituents." Sections of extreme thinness must be used, as the dolomite crystals are often less than 10 μm in size. Walker also mentions that thin sections of reacted material will reveal changes in the euhedral dolomite crystals, either through alteration to calcite or the crystal may be partially or completely removed.

Others have suggested that although no characteristic gel is produced, the reaction product formed through ACR, the presence of brucite ($\text{Mg}(\text{OH})_2$) or more complex magnesium silicates can be used to assist in diagnosis (ACI 1998). Brucite, also known as magnesium hydroxide, can be identified petrographically (Farny and Kosmatka 1997) or through the use of an SEM, although it is difficult to observe, and therefore its absence should not be taken as evidence that ACR is not present (ACI 1998).

Diagnostic Features for External Sulfate Attack

It is not possible to positively identify sulfate attack through visual means alone, although visual assessment of a pavement can contribute to the diagnosis. Map cracking, particularly in the vicinity of joints, the observation of a white precipitate, evidence of concrete expansion, and concrete paste disintegration are all signs of external sulfate attack. In the case of external sulfate attack from impurities in chemical deicers, a polygonal crack pattern throughout the pavement becoming more concentrated as it approaches the joint is common (Wolter 1997). Unfortunately, these descriptions are similar to visual cues for other types of MRD as well. It is therefore necessary that concrete specimens be obtained for laboratory evaluation to positively identify the distress mechanism.

Site investigation can reveal conditions that may contribute to sulfate attack. Factors influencing sulfate attack are the amount and nature of the sulfate found in the soil, the level of the water table and its seasonal variations, the sulfate content of the groundwater, the flow of groundwater, the soil porosity, and the type of construction (Mehta and Montiero 1993). Also, if impurities in the deicer salts are suspected, the source of the deicer should be tested.

Barium chloride potassium permanganate (BCPP) stain can be used to detect the presence of sulfate minerals, such as ettringite, gypsum, and anhydrite phases, in concrete (Poole and Thomas 1975). The principal staining mechanism is a two-step process. The concrete is initially immersed in the stain solution, causing sulfate ions released from the concrete to precipitate as barium sulfate. As this occurs, the potassium permanganate co-precipitates with the barium sulfate imparting the characteristic "purple" color to the resulting crystal. This process results in a permanent alteration of the sulfate phase surface at the sulfate/water phase boundary. The concrete is then rinsed in a saturated solution of oxalic acid to remove any surface coloration from precipitation of excess potassium permanganate. The remaining purple colored crystals identify where ettringite, gypsum, or anhydrite phases existed. This method is

particularly useful in identifying the presence of sulfate minerals that may have filled air voids, since small filled air voids are at times difficult to distinguish from hydrated cement paste using the stereo OM.

Chemical analysis can be conducted on distressed concrete suspected of suffering external sulfate attack to determine if more acid soluble sulfate is present than was contained in the original cement. If external sulfate attack is responsible, the sulfate content will be many times that derived from the normal calcium sulfate content of the cement. But in some cases, the excess sulfate may have been leached from the concrete, and therefore this test alone cannot be used to conclusively reject external sulfate attack as the primary distress mechanism. Testing for acid-soluble magnesium determines if concrete has suffered magnesium sulfate attack (Mielenz 1962).

Petrographic analysis can be used to detect symptoms of sulfate attack. As described previously, ASTM C 856 provides some diagnostic features for ettringite that can be used to identify its presence within the concrete. Crystalline ettringite occurs as fine, white fibers or needles or spherulitic growths in voids, in the cement paste or lining fractures (ASTM C 856). Marusin (1998) asserts that patches of "gel like" ettringite are often agglomerations of fine ettringite needles. External sulfate attack may manifest itself as an expansion of the concrete cement paste matrix. In the intermediate stages, the reaction produces great expansion in the cement paste, resulting in progressive distention, warping, and fracturing of the concrete. The cement paste tends to expand away from the aggregate particles, and the resulting gaps can become filled with secondary ettringite deposits. Advanced sulfate attack results in soft, friable cement paste, and some of the aggregate rock types may also be altered. The outer hull of highly carbonated mortar remains as a shell until finally broken by expansion of the underlying concrete (Mielenz 1962).

Given the absence of any abnormal internal sources of sulfate, the presence of ettringite alone is not evidence of external sulfate attack (Thaulow and Jakobsen 1997). Ettringite is a normal hydration product of portland cement. Although ettringite crystals found in air voids or fractures may be responsible for the fracture, it is just as likely that the ettringite may have precipitated out in the space afforded by the air voids or fractures. There has been a lack of correlation shown between the amount of observed ettringite filling voids and cracks and the amount of expansion, and there is also believed to be more than one type of ettringite, and not all are expansive (DePuy 1994).

Thaulow and Jakobsen (1997) list the following three microscopic features that can be used in the diagnosis of external sulfate attack:

- Surface parallel cracks in the cement paste and along aggregate-paste interfaces.
- The presence of gypsum and/or excessive amounts of ettringite in voids, cracks, and paste.
- The dissolution and decalcification of cement paste.

Surface parallel cracks run parallel to and near the concrete surface, and are commonly filled or partly filled with gypsum. The cracks typically traverse through the cement paste, following aggregate surfaces. According to Thaulow and Jakobsen (1997), gypsum can be distinguished by its texture and birefringence when observed in crossed polarized light. If the gypsum has precipitated in the paste, an SEM with an EDS will be useful in identification. The presence of surface parallel cracking must be present if external sulfate attack is the mechanism at work, as it is an indicator of near-surface paste expansion. Thaulow and Jakobsen also emphasize that an external source of sulfate must be present.

Bickley et al. (1994) describe other techniques that can be used to identify the type of deterioration, its cause, and the type of cement. Chemical analysis by Inductively Coupled Plasma (ICP) spectrophotometry, LECO combustion, ion chromatography, microstructural/microchemical examination by scanning electron microscope with energy dispersive spectrometry (SEM-EDS), mineralogical/phase analysis by x-ray powder diffractometry (XRD) and thermogravimetric analysis/differential thermogravimetry (TGA/DTG) can be used on the concrete samples. Characteristic XRD spacings for thaumasite occur at 0.956 nm and 0.552 nm, while those for ettringite are at 0.973 nm and 0.561 nm.

Diagnostic Features for Internal Sulfate Attack

Internal sulfate attack is even more difficult to diagnose than external sulfate attack. Map cracking, particularly in the vicinity of joints, the observation of a white precipitate, evidence of concrete expansion, and concrete paste disintegration are all signs of potential sulfate attack. Unfortunately, these same manifestations are consistent with other distress mechanisms as well.

It is useful to determine the total acid soluble sulfate content of the concrete to determine if it is significantly greater than that present in the original concrete. If excess sulfate concentrations are measured, it indicates that the source of sulfates is likely external. On the other hand, if the sulfate content is similar to the expected total sulfate content of the concrete, the sulfate attack may be from internal sources of sulfate. But in some cases, the excess sulfate may have been leached from the concrete and thus this testing cannot be used to conclusively establish that internal sulfate attack is at work.

Internal sulfate attack due to delayed ettringite formation can be identified as an expansion of the concrete cement paste matrix. This is characterized by peripheral cracks or gaps around the coarse aggregate particles (Skalny 1996). The cracks around the aggregates are proportional in width to the size of the aggregate, consistent with a linear expansion of the cement paste. In the intermediate stages, the reaction produces great expansion in the cement paste, resulting in progressive distention, warping, and fracturing of the concrete. The cement paste tends to expand away from the aggregate particles, and the resulting opening can become filled with secondary ettringite deposits. The identification of early DEF related cracks in the cement paste is difficult given that the cracks are often small, requiring an SEM for identification. The question then arises as to the source of the crack (i.e., is the cracking the result of paste expansion or a result of the vacuum desiccation of the specimen in the SEM?). The use of a LVSEM or ESEM would be helpful in this regard.

Gaps between the paste and aggregate are thought to be one of the most distinguishing microscopic features of concrete affected by DEF, a form of internal sulfate attack (Thaulow and Jakobsen 1997). The gaps may be empty, partially filled with crystalline ettringite, or with a massive, poorly crystalline ettringite. Although some have speculated that the growth of ettringite created the gaps, Thaulow and Jakobsen (1997) believe that the gaps first formed due to expansion of the paste and secondary ettringite formation later filled them. It is noted that gaps can also form around aggregates due to paste expansion resulting from freeze-thaw of critically saturated paste.

Another distinguishing feature of internal sulfate attack is that, in contrast to external sulfate attack, gypsum is not normally formed during DEF (Thaulow and Jakobsen 1997).

Diagnostic Features for Corrosion of Embedded Steel

There are several methods available to determine the extent to which a concrete structure has undergone corrosion of reinforcement and the secondary deterioration that has been caused by corrosion. The simplest method is to conduct a survey using a trained professional to examine the structure and document the location of cracks, spalls, delaminations, and other features of deterioration (Perenchio 1994; Erlin and Hime 1987). Cracks on the surface over reinforcing bars, spalls with steel reinforcement at the bottom, delamination, and rust colored staining are all signs of corrosion distress.

Chapter 4 of ACI 222R provides an excellent summary of diagnostic procedures available for identifying corrosive environments and active corrosion in concrete (ACI 1989). In addition to visual surveys, one recommended tool is a pachometer that can be used to locate steel and determine the thickness of the concrete cover. The thickness of the cover is important as it has a direct influence on whether corrosion is likely to develop in a corrosive environment.

A delamination survey is recommended as part of the survey, in which delamination detectors are used to sound out the concrete surface. This survey consists of striking the concrete surface with a metallic object such as a rod, hammer, or chain. More sophisticated equipment such as a Delamtect can be employed to complete a thorough survey (ACI 1989). Concrete that is of lesser quality or damaged will make a hollow sound when hit with the object (Perenchio 1994). Visual and delamination surveys are used together to determine the extent of potential corrosion distress, identifying areas that are candidates for an in-depth study.

There are two wet chemical analyses that are commonly performed on concrete to determine the chloride ion concentration. One is used to determine water-soluble chloride (ASTM C 1218) and the second to determine acid-soluble chloride content (AASHTO T 260). In each case, a concrete specimen is obtained and ground to powder, typically using a rotary hammer to avoid the use of cooling fluids. The most common test is the acid solubility test, in which the pulverized concrete is dissolved in nitric acid. The water-soluble method has seen limited use because it is sensitive to several factors, including time of leaching, temperature, and degree of grinding. For the same concrete, the average chloride content measured by the water-soluble method is typically 75 percent to 80 percent of its acid-soluble chloride content (ACI 1989).

Chloride content is most often expressed as the chloride ion (CL) present as a percentage of the mass of cement. It is recognized that the cement content of in-service structures is not always known so, alternatively, the chloride content can be expressed in terms of percent or weight of chloride ions per volume of concrete. It is very important that the chloride content being reported is defined according to the method of testing.

Concrete cores may also prove useful in determining the extent to which a structure has undergone corrosion of reinforcement. Cores can be used to verify whether delamination or debonding of the concrete and steel has occurred. Concrete cores can also be used to determine the depth of the steel to evaluate whether adequate cover was present. If dowel lockup is suspected after measuring joint movement, coring through a dowel can verify whether it can still move freely. A petrographic evaluation of concrete cores can also be used to reveal if the void structure is adequate and whether other deleterious mechanisms are at work in addition to corrosion.

Carbonation can be a contributing factor in corrosion of embedded steel and therefore it is useful to determine the depth of carbonation. Determination of the depth of carbonation can be accomplished using phenolphthalein stain or other pH indicator, since carbonated cement paste will have a pH significantly lower than that of unaltered cement paste.

Approach to Data Analysis and Interpretation

The interpretation and diagnosis of MRD relies primarily on information collected during laboratory investigation, supplemented with information collected during the review of the records and visual assessment of the pavement surface. When diagnosing a concrete distress, often there is no clear answer as to which distress mechanism caused the failure as multiple mechanisms are observed. This makes it difficult to determine which mechanism(s) might be responsible for the initial deterioration versus those that occurred after the fact as opportunistic distress mechanisms. Various types of distress mechanisms can occur simultaneously in concrete and each can incrementally contribute to the ultimate failure of the material. This fact must be taken into account when evaluating MRD in concrete pavements.

In approaching laboratory diagnosis of MRD, the analyst must put aside preconceived notions as to what the MRD might be. Instead, diagnosis should be approached through systematic data collection, linked to a process of elimination. A general philosophy of "asking the material questions" must be adopted where the analyst determines which diagnostic features are identifiable within the concrete. For example; "Are there microstructural features indicating AAR?" or "Is the air-void system adequate for the concrete

service conditions?” After examining the concrete and noting all available information, the analyst can only make an educated judgment as to why the material failed. In some cases, there will be indications of a clear cause while in other cases, there may be multiple mechanisms at work making it difficult to determine precisely which factor is primarily responsible.

The approach taken, as presented in the guideline in Volume 2 of this Final Report, is to use a series of four flowcharts to systematically guide the analyst through the evaluation. The analyst inspects the concrete using the methods described in the previous sections, guided by the hierarchy of questions presented in the flowcharts. The responses to the questions presented in the flowcharts determine what analytical procedures will be performed. As the analyst moves through the flowcharts, there is the potential for more than one MRD being identified.

In the procedures described in the guideline, the analyst first performs a visual inspection of the concrete to assess its general properties. In most cases, visual inspection helps build a hypothesis of failure, but does not itself directly lead to the cause of distress. Most of the observations will be confirmed at a later stage using optical microscopy or an SEM examination. However, there are some general conditions, with visual diagnostic features, known to contribute to, or directly cause, distress.

After completing the visual inspection, the concrete is examined with the stereo OM and/or staining techniques to assess the paste and air-void system. Quantitative measurements of the air system parameters should be obtained using methods consistent with ASTM C 457. The process may continue, analyzing the paste fraction using the petrographic OM, the SEM, chemical analysis, and/or the XRD.

Having completed the stereo OM and staining evaluations, the analyst can begin the process of assessing the condition of the aggregate structure. Aggregate quality is fundamental to concrete performance and any degradation of the aggregates should be closely scrutinized. To complete this analysis, it may be necessary to use the petrographic OM, SEM, chemical analysis, and/or XRD.

The final flowchart presents the procedure for identifying in-filling material. The identification is accomplished by using staining techniques, petrography, and/or x-ray microanalysis using the scanning electron microscope. The identification is straightforward using staining techniques, yielding a yes/no answer whereas x-ray microanalysis yields a full chemical analysis that can be used to identify the specific material. Optical petrography can also be used, but requires more skill and training, as the optical properties are more difficult to discern. ASTM C 856 includes a complete table of common minerals found in PCC and their optical properties. Regardless of the composition of the in-filling material, the analyst must consider the following two things to confirm the MRD type based upon identification of the in-filling material:

- The presence of secondary deposits alone does not mean the associated MRD is the cause of the observed distress. The presence or absence of other diagnostic features to positively identify the associated MRD must be verified.
- A cause and effect relationship must be established between each possible MRD and the observed distress. For example, the diagnostic feature of ettringite deposits in air voids, by itself, is not evidence of sulfate attack. There must be other evidence such as cracking, paste expansion, or gypsum corrosion. Also, secondary deposits of ettringite are commonly thought not to be expansive. In the end, more evidence other than ettringite in the air voids is required before identifying sulfate attack. The same caution must be exercised for all MRD types.

As the analyst works through the flowcharts to complete the laboratory analyses, he/she notes all possible MRDs identified and then consults tables that summarize the common diagnostic features associated with each MRD. These tables summarize the principal, common, diagnostic features for each

MRD, as characterized by laboratory methods and as seen during field evaluation. Included in these tables are the diagnostic features, methods of characterization, and specific comments relative to either the observed characteristic or the test method. The tables are intended to serve as a quick reference during the diagnosis of MRD by assisting the analyst in identifying other diagnostic features to help confirm the presence of a particular MRD. The tables also serve to tie together the results of visual inspection performed in the field, with laboratory inspection and diagnosis.

In the end, it is not always possible to identify a single MRD as the cause of the observed distress. This conclusion should not be viewed negatively, but instead as a recognition that on many occasions more than one MRD may be active in a distressed concrete pavement, making absolute identification of the primary distress mechanism difficult or impossible. In such cases, the analyst should list all possible MRD mechanisms and assign a relative rating as to the likelihood of each being responsible for the observed distress. The rating scheme should be simple and subjective, possibly along the lines of a scale ranging from highly unlikely, unlikely, possible, probable, to highly probable. In this way, the analyst can present to the engineers and other interested personnel what he/she thinks is the most likely cause(s) of distress while still presenting all possibilities. In the end, this will help focus the repair/rehabilitation efforts and preventative strategies for future construction without turning a blind eye to other possible causes.

Summary on Data Analysis and Interpretation

The procedures presented in the guideline contained in Volume 2 of this final report are intended to lead the analyst through identification of common MRD types based upon typical diagnostic features. Although one MRD may be present, the flowcharts have the analyst examine diagnostic features for all MRD types, thereby minimizing the probability of prematurely "zeroing in" on a single MRD without considering all possibilities. The flowcharts do not address every possible combination of MRD nor do they address every set of possible diagnostic features that may be seen in distressed concrete. It should be understood that the mechanisms responsible for MRD are complex and may manifest themselves differently under different conditions and the exact nature of a given distress may vary.

It is very common to observe multiple distress mechanisms at work when examining distressed concrete. When multiple distresses are present, it is clearly more difficult to diagnose an absolute cause of failure and the analyst should be cautious when trying to do so. Such an attempt will almost invariably require some assumptions of the order, magnitude, or cause of the earliest failures. Likewise, analysts and engineers are cautioned from trying to diagnose MRD based on laboratory results alone. However, a complete laboratory analysis, when combined with field evaluation data and construction and service records, will help accurately describe the condition, environment, and performance of the concrete pavement in question. From this type of broad-based evaluation of a MRD problem, conclusions about the source of distress can often be reached.

3.3 Background for Treatment and Prevention of MRD in Concrete Pavements

To this point, this Final Report has focused on the identification of MRD in deteriorating concrete pavements. In an ideal world, material testing and other preventive measures would eliminate the occurrence of MRD. However, as evidenced by the many miles of pavements exhibiting durability problems, it is difficult to correctly identify and address all potential durability problems. Hence, means of addressing such problems need to be established and understood. And due to the limited effectiveness of treatment and rehabilitation strategies, it is imperative that strategies are implemented to prevent MRD in newly constructed pavements.

Background on Treatment and Rehabilitation Methods

Materials-related problems are becoming increasingly identified as the cause of concrete pavement deterioration. In many cases, pavements affected by MRD would likely be allowed to deteriorate significantly before the MRD is identified, leaving replacement of the deteriorated concrete as the only option to restore serviceability. However, a few methods that allow the treatment of in-place materials are now available. These treatments have evolved from a better understanding of the mechanisms of MRD and from improved techniques that permit early identification of MRD. As diagnostic technologies improve, better and more timely treatment methods are sure to follow.

Methods for addressing MRD can be broadly categorized as either treatment methods or rehabilitation methods. Treatment methods are defined as those methods designed to prevent further development of the distress or to reduce its rate of progression. Examples of treatment methods range from the application of chemicals to alter adverse reactions to the use of joint and crack sealants to prevent excess water infiltration. Rehabilitation methods, on the other hand, are defined as those methods designed to remove deteriorated areas and to maintain serviceability of the pavement. Such methods include patching of small, deteriorated areas or more comprehensive methods such as overlays or reconstruction.

This section presents the available treatment and rehabilitation methods for pavements with MRD and the recommended approach for treatment developed in this study. The discussion within this section includes applicability of the available methods and materials, current practices, laboratory and field performance results, and specific recommendations.

Available Treatment Methods

Treatment methods include chemical treatments, joint and crack sealing, crack filling, surface sealing, retrofitted drainage, drying, and restraint. The purpose of the various treatments is either to arrest the mechanisms that cause the distress or reduce or inhibit the progression of the distress mechanism. The different treatment methods work in a variety of ways. For example, chemical treatments affect the reactions that cause the distress. Retrofitted drainage, on the other hand, provides its effectiveness by limiting the amount of available water in the pavement system that would otherwise exacerbate the distress. To be effective, treatment measures need to be performed during the early stages of the deterioration process. If freeze-thaw deterioration has progressed to the moderate-severity level, treatment methods will no longer be effective, and rehabilitation methods must be used instead.

Chemical Treatments

Chemical treatments are often used in fresh concrete to prevent reactions and, in some cases, can also be effective as treatments on existing concrete pavements. The main drawback of applying such treatments to existing pavements is the difficulty in achieving penetration of the chemical treatment to a sufficient depth in the slab to where it is effective. With fresh concrete, the chemical treatment can simply be mixed with the other constituents and spread throughout the concrete. Since the treatment can only be applied to the surface of existing pavements, to be effective the chemical must be able to penetrate the surface. While multiple applications can increase the depth of penetration, such treatments will never be able to penetrate the entire depth of the slab.

Chemical treatments are designed to mitigate the destructive reaction that causes the distress. On existing concrete pavements, chemical treatments have been used to:

- Arrest the existing reaction.
- Alter the existing reaction.
- Activate another reaction to offset the existing reaction.

The common component to all of these treatments is a chemical reaction. As a result, chemical treatments are only effective on distress types that are caused by reactions, such as ASR and sulfate attack. Other distress mechanisms, such as freeze-thaw deterioration and salt scaling, do not involve a reaction and thus cannot be addressed through chemical treatments.

An example of one of the more promising chemical treatments is lithium salts, which can be used as a treatment for addressing ASR. Lithium salts were first found to be an effective additive to fresh concrete to prevent abnormal expansion due to ASR (Stark et al. 1993). They have since been found to be effective in laboratory testing of concrete samples and are beginning to see limited use in the field, although they have yet to be proven effective for widespread use. There are currently several on-going experimental projects being conducted to evaluate the effectiveness of lithium salts, and early results have been favorable (Stark et al. 1993; Johnston 1997). As previously noted, the major limitation for field applications is achieving penetration of the lithium solution through the full depth of the concrete slab.

One study tested a series of specimens, which included variations in the amount of expansion allowed before treatment, the type of treatment solution, and patterns of soaking and drying (Stark et al. 1993). The addition of lithium solutions into hardened mortar exhibiting large expansion due to ASR was found to reduce further expansion, whereas the control specimens continued to expand (see figure 1-7). Of the three types of treatment solutions, LiOH solutions were more effective in controlling expansion than either Li_2CO_3 or LiF solutions. As shown in figure 1-8, the expansion nearly leveled off after the sample was soaked in the LiOH solution.

However, it should be noted that the long-term effects of lithium salts have not been studied; the laboratory tests were only 25 months long. Preliminary results from another study also show signs that lithium salts are effective in field applications (Johnston 1997). That study avoided the use of LiOH due to safety concerns and the ability of OH ions to exacerbate the ASR. More recent successes in field applications have been reported at a workshop sponsored by the ACI and the FHWA entitled, *Concrete Durability: ASR and Other Deterioration Mechanisms*. At the pilot workshop held in Baltimore on September 11 & 12, 2000, David Stokes of the FMC Corporation reported on successful field applications of lithium nitrate on existing pavements in an attempt to arrest expansion due to ASR. Long-term monitoring of the cited test sites have not yet been done, so the effectiveness of the treatment could not be evaluated.

The addition of chloride ions offers a potential treatment method for sulfate attack regardless of the source of sulfate ions. Ettringite has been found to dissolve in the presence of chloride ions, particularly NaCl (Attigbo et al. 1990; Marks and Dubberke 1995). Laboratory testing of concrete cores containing ettringite confirmed that treatment with NaCl can dissolve ettringite. However, this process initially involves further expansion of ettringite before it dissolves. Of particular concern is the potential harmful effect that chloride ions can have on the pavement, as NaCl can result in scaling of the concrete surface as well as corrosion of embedded steel. Further, as described previously under deicer attack, salts can also result in the dissolution of calcium hydroxide, which may increase paste porosity and thus permeability.

Chemical treatments often require special mixing, curing, and application techniques. The manufacturer's guidelines for each product should be followed closely. When using a chemical treatment, an important consideration is achieving penetration of the chemical into the concrete. Achieving penetration through the full depth of the concrete is impossible, but careful application techniques can help increase the effectiveness. Thorough application of the chemical around and within joints and cracks, where MRDs are often concentrated, can be beneficial. Repeated application of the chemical treatment can also help increase the effectiveness of the treatment. Another important consideration when using chemical treatments is safety. Products that can be potentially harmful should contain special safety considerations; the manufacturer should be contacted if safety guidelines are not clearly identified.

Joint and Crack Sealing

Moisture is key to the development of many durability-related distress types. As a result, these distress types are often observed to be much more severe at joints and cracks, where moisture can penetrate the pavement surface. Sealing joints and cracks can help reduce the intrusion of moisture into the pavement. However, it should be recognized that it is impossible to completely eliminate the intrusion of moisture into joints and cracks, especially for an extended period of time. For example, a Minnesota study found that the amount of infiltrated water (measured using tipping buckets at drainage outlets) returned to the same levels within 2 weeks after resealing (Hagen and Cochran 1995).

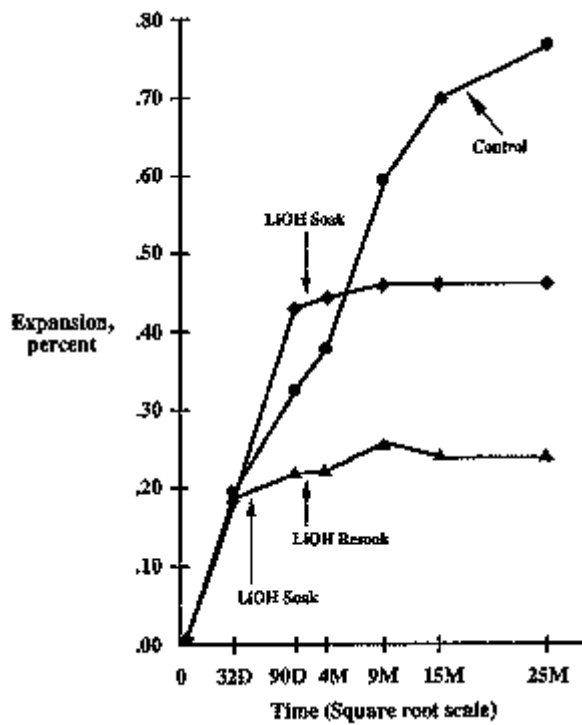


Figure 1-7. Effect of treatment with LiOH solution (Stark et al. 1993).

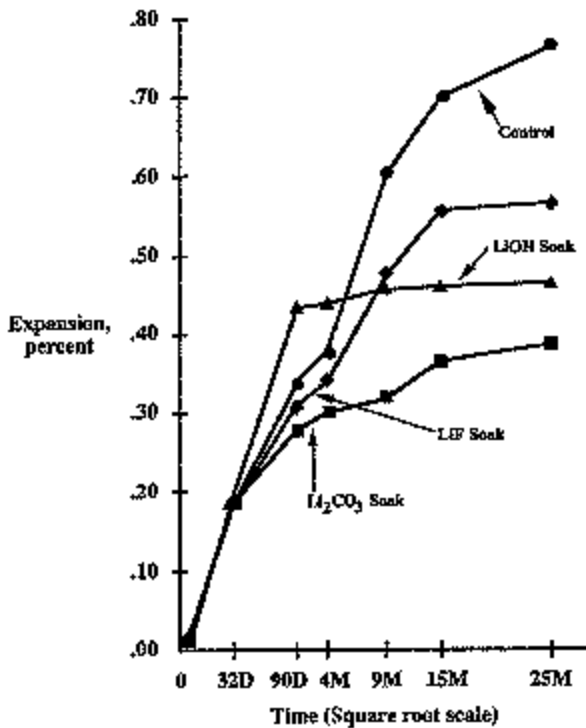


Figure 1-8. Effect of various lithium treatments (Stark et al. 1993).

Consequently, many designers would argue about the effectiveness of any method proposed to eliminate moisture. Although the amount of water from surface infiltration can be significantly reduced, it can never be completely eliminated. In addition, the underlying subgrade remains continually moist, even when the most elaborate drainage system is used. Furthermore, water can also enter the pavement from other sources, such as laterally from ditches and upward from the groundwater table. Nonetheless, it is still believed that there is some value in taking measures to reduce the amount of available moisture. However, such methods should be used with caution and are recommended only under certain conditions.

Sealing of joints and cracks is most effective at reducing distresses that initiate at these discontinuities. Freeze-thaw deterioration of aggregate (D-cracking) is a prime example. Freeze-thaw deterioration of aggregate typically initiates at joints and cracks where water is allowed to infiltrate, so it only makes sense that measures taken to prevent the intrusion of water can be effective. Sealing joints and cracks will limit the amount of infiltrated water. However, the effectiveness of sealing depends on several issues:

- How much water will be prevented from infiltrating by sealing the joint or crack?
- What other sources are contributing moisture and what is the volume of water from other sources?
- How long will the sealant be effective at reducing the infiltration of water?

Sealing joints and cracks will reduce the amount of infiltrated water, although it will not completely eliminate infiltration. The effect (if any) that this reduction in moisture infiltration will have on reducing freeze-thaw deterioration is debatable. Studies have shown that even one freeze-thaw cycle can be as damaging as multiple cycles (Janssen 1985). To be totally effective, sealing must totally eliminate the infiltration of moisture so that even one saturated freeze-thaw cycle is prevented.

Sealing joints and cracks can also help reduce the penetration of sulfate ions that are responsible for sulfate attack. This method is only viable if the sulfate ions are from an external source applied to the surface; it does not provide any benefit if the source of the sulfate ions is from within the portland cement or from the underlying subgrade. As a result, methods to limit the amount of available water in the pavement system are not as effective as other treatment methods.

Crack Filling

Crack filling must first be differentiated from crack sealing. Crack filling refers to the filling of surface cracks (e.g., map cracking) with a material that penetrates into cracks, and not the sealing of individual full-depth transverse and longitudinal cracks. The purpose of crack filling is not to prevent intrusion of moisture and incompressibles but rather to strengthen the concrete pavement. Crack fillers penetrate into cracks and effectively "glue" the concrete pieces together.

An example of a crack filling material is high molecular weight methacrylate (HMWM). HMWM strengthens the concrete by filling the cracks and bonding the cracked concrete pieces together. Such treatments have been most effective when applied to cracks that are wide enough for the material to penetrate (Engstrom 1994). Freeze-thaw deterioration of aggregate and ASR are examples of distress types that might be effectively treated with crack fillers.

Although the cracking associated with freeze-thaw deterioration is not confined to the surface, crack fillers can still provide some benefit. Crack fillers should only be applied at joints and cracks where freeze-thaw deterioration occurs; there are no benefits to applying them to the entire pavement surface. Field experiments found that HMWM were effective for up to 18 months, which implies that reapplication at such intervals may be required (Engstrom 1994). Several products were tested in the field experiments; the results revealed that Transport T70X and 3M 4R Concrete Restorer performed better and were less costly than the Sika product (Engstrom 1994). Cores taken at treated areas indicate that HMWM penetrated cracks up to 75 mm deep.

HMWM has also been used successfully on pavements affected by ASR. The map-like cracking pattern produced by ASR typically extends only 50 to 75 mm below the surface. In one study, cores taken on State Route 58 near Boron, California found the HMWM penetrated cracks up to 50 mm deep, the maximum depth of the surface cracks (Stark et al. 1993). On an experimental project on I-80 near Winnemucca, Nevada, significantly lower midslab and joint deflections (11 and 44 percent, respectively) were measured just one day after HMWM application (Stark et al. 1993).

Crack filling with an HMWM requires some special considerations (Engstrom 1994). First, all bituminous patches should be removed because HMWM deteriorates the asphalt. In addition, all cracks should be thoroughly cleaned by airblasting in order to help achieve penetration into cracks. Finally, the treatment should be covered with sand within 20 minutes after application in order to ensure good skid resistance.

Surface Sealing

Surface sealing or coating helps prevent the ingress of moisture into the pavement, which can prevent initiation or limit the extent of moisture-induced distress. This treatment method forms a penetration barrier on the pavement surface, which expels moisture much like wax on a car. Sealers can also reduce or prevent the ingress of oxygen, carbon dioxide, chloride ions, sulfate ions, and other constituents that contribute to damaging reactions.

Concrete sealers can be divided into two categories—coatings and penetrants. Coatings form a film on the pavement surface, whereas penetrants are designed either to fill the pores or line them with a water-repellent substance (Campbell-Allen and Roper 1991). Examples of surface sealers include silane sealants, penetrating oils, and two-part resins.

Sealers have proven to be effective in laboratory testing where concrete samples can be sealed on all sides. However, as with chemical treatments, sealers can only be applied to the surface of existing pavements. Moisture and other constituents can still penetrate the concrete vertically through the bottom and laterally through the sides of the slab. Sealers are thus most effective at limiting constituents that infiltrate from the pavement surface, such as chloride ions from deicing salts. Surface sealers are more effective when used in conjunction with other methods designed to address moisture-related distress.

As with other methods designed to eliminate the intrusion of moisture and deleterious constituents, concrete sealers can never be totally effective. Although they will significantly reduce the amount of water that penetrates the surface, water can still enter the pavement from other sources. The usefulness of surface sealers is more questionable because some MRD, such as freeze-thaw deterioration of aggregate, initiate at the bottom of the slab and propagate upward.

Nonetheless, surface sealers have been used to combat the effects of freeze-thaw deterioration. They have been more effective in laboratory tests (where the sealer completely coats the sample) than in field experiments. Laboratory testing (later confirmed through field testing) found that water-based and solvent-based silane sealers slowed the rate of deterioration, whereas penetrating oils and two-part resins were not as effective (Janssen and Snyder 1994). Another field experiment using silane sealers indicated mixed results, although they were found to be more effective on pavements with less deterioration (Engstrom 1994).

Surface sealers have proven to be more effective for addressing freeze-thaw deterioration of the cement paste. Unlike freeze-thaw deterioration of aggregate, where deterioration is worse at the bottom of the slab, freeze-thaw deterioration of cement paste typically results in more damage at the pavement surface.

Sealing the pavement can also help prevent the penetration of sulfate ions from external sources. Practically, however, only the pavement surface can be completely sealed, and the external sources of sulfate ions (groundwater, soils) make contamination of the bottom of the slab more likely. For this reason, the benefits of surface sealing are questionable for external sulfate attack.

The use of surface sealers for controlling ASR is even more questionable because studies have shown that even water in the vapor phase (relative humidity greater than 80 percent) is sufficient to cause swelling of the gel product (Stark et al. 1993). In one study, the use of a silane surface sealer was found to have little to no meaningful effect (Stark et al. 1993). Although the surface sealer did prevent moisture transfer in the liquid phase, it did not prevent moisture transfer in the vapor phase.

For salt scaling, the only feasible treatment method is the application of a surface sealer. Elimination of the destructive component, namely the chloride ions within deicing chemicals, is not a possibility because they are required for safety (although a nonchloride deicer could be used instead). Surface sealers must prevent the ingress of moisture and chloride ions into the pavement without disrupting the safety characteristics of the pavement. This method should only be used for pavements exhibiting limited amounts of low-severity scaling unless previous steps are taken to address the higher severity scaling.

As with chemical treatments, the composition of surface sealers is continually changing as further advancements are made (Campbell-Allen and Roper 1991). The manufacturer should be consulted to ensure that the appropriate products are used and that proper application techniques are followed. A clean, dry surface is required to ensure good bonding and thorough penetration of the sealer. Airblasting or sandblasting may be used for this purpose. Diamond grinding is a good technique to provide these surface properties, as well as to remove surface irregularities. Traffic should not be allowed on the pavement until the sealer has fully penetrated the pavement and evaporated. For silane sealers, a delay of 20 to 45 minutes is typically required (Engstrom 1994).

Retrofitted Drainage

The addition of retrofitted drains will, in theory, remove excess moisture from under the slab and at joints and cracks, which would assist in slowing or delaying MRD. In reality, however, water cannot readily move through a dense-graded base (typically found in many older, deteriorating pavements) to the retrofitted drains at the edge of the pavement. Therefore, the effectiveness of retrofitted drains is reduced, and other means of moving water from the underlying layers to the drains must be provided.

Moisture-induced distresses generally initiate and progress more quickly at the bottom of the slab, which is exposed to moisture for prolonged periods. Because the bottom of the slab is not exposed to certain environmental effects, even a light rain can saturate the underlying layers, which will then remain saturated for prolonged periods. Providing a means to remove moisture at the slab-base interface will help shorten the time the pavement is exposed to moisture. An important consideration is the permeability of the layers beneath the slab. Studies have shown that if water cannot move from within the underlying layers to the retrofitted drains, the drainage system will be only marginally effective at removing water and even less effective at reducing moisture-related distress (Smith et al. 1996). Consequently, retrofitted drainage should only be considered where water can move to the drain, and even then its effectiveness is questionable.

In order to be effective, retrofitted drainage must be applied during the early stages of deterioration. By the time moisture-related distresses are apparent on the surface, deterioration at the bottom of the slab has often progressed to the point where retrofitted drainage will no longer be effective.

Drying

As the term implies, drying refers to completely removing moisture from the concrete pavement. Moisture plays a key role in many durability problems, so drying of the concrete will delay the progression of such distresses. Unfortunately, it is impossible to keep water out of the pavement, and the pavement will eventually become resaturated. Nonetheless, complete drying can offer prolonged benefits by altering the reaction or changing the reaction products.

Drying has been found to produce some beneficial and irreversible effects for some durability problems. A prime example is ASR, in which a substantial portion of the alkali hydroxide in the pore solution becomes fixed upon drying and does not return to solution even after resaturation (Stark et al. 1993). The beneficial effects of drying can also be long lasting. Laboratory studies have shown that the rate of ASR is substantially diminished by drying even with prolonged soaking and will never be completely reversed (Stark et al. 1993). This phenomenon, which was discovered by accident but later confirmed through laboratory testing, is believed to be the result of carbonation. The major shortcoming of this technique is that there is no practical way to sufficiently dry pavements in the field.

Restraint

Restraint can be used to combat the expansive forces developed as a product of some durability problems. Both ASR and sulfate attack produce reaction products that are larger in volume than the original products, thus causing expansion and cracking of the concrete pavement. Restraint induces internal compressive stresses in the concrete that prevent the development of cracking, in much the same manner as a prestressed concrete pavement.

Physical restraint has shown some success in controlling the expansive forces produced by ASR. Three-dimensional restraint has proven effective for controlling expansion in concrete columns but is not feasible for highway pavements. Laboratory testing on miniature pavement sections revealed low levels of one-dimensional restraint helped control microcracking but higher levels actually accelerated

expansion and internal cracking (Stark et al. 1993). The study also concluded that one-dimensional restraint has a significant positive effect on transverse and vertical strain; finite element modeling confirmed that one-dimensional restraint has a positive influence of restraint in three dimensions (Stark et al. 1993).

The major shortcoming with the use of restraint in actual pavements is that it is extremely expensive and that (in order to be effective) the restraint must be applied early in the life of the pavement before the extent of potential damage has been observed. For pavements, three-dimensional restraint is not feasible; uniaxial restraint offers the only practical approach, yet it is still an expensive proposition. The degree of restraint is also an important consideration, as too little restraint will not prevent cracking perpendicular to the direction of restraint and too much restraint can cause cracking parallel to the direction of restraint.

Available Rehabilitation Methods

Rehabilitation methods, as opposed to treatment methods, follow a totally different approach to addressing MRD. Rather than altering the development or progression of MRD, their purpose is merely to repair deteriorated areas to maintain serviceability and possibly extend the life of the pavement. Rehabilitation methods include partial- and full-depth repairs, slab replacement, diamond grinding, overlays, reconstruction, and recycling. Some pavement engineers believe that rehabilitation methods are not effective means to repair MRD because they do not address the cause of the distress. However, when used under the right conditions (such as to maintain the serviceability of the pavement), repair methods can prove cost effective.

Partial-Depth Repairs

Partial-depth repairs are one rehabilitation method that can be used to repair localized deteriorated areas caused by MRD. These repairs consist of the removal of concrete near the surface and replacement with an acceptable patch material, usually 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 of the concrete slab. Partial-depth repairs are most commonly performed along transverse and longitudinal joints.

Examples of MRD that are candidates for partial-depth repairs are freeze-thaw deterioration of cement and deicer scaling/deterioration. These deterioration mechanisms typically result in scaling, crazing, or map cracking at the pavement surface. For partial-depth repairs to be cost effective, the deterioration must not be too widespread. Deteriorated areas requiring multiple partial-depth repairs can be repaired more cost effectively through other rehabilitation methods, such as full-depth repairs or overlays.

Furthermore, the deterioration must be limited to the upper one-third of the slab. Cores taken at representative areas of deterioration can help determine the extent of deterioration for advance planning. Before placing the patch material, the patch area should be "sounded" with a hammer to ensure that all deterioration has been removed. When damage is found during construction to be more extensive than anticipated, the repair area should be expanded or converted to a full-depth patch, if necessary. The repair will not perform adequately if the deterioration is not completely removed.

Partial-depth repairs are not recommended for MRDs that extend beyond the upper third of the slab. Some MRDs initiate and deteriorate more rapidly at the bottom of the slab where moisture and deleterious chemicals are more readily available. Freeze-thaw deterioration of aggregate and external sulfate attack are examples of such distress types. Other MRD, such as ASR and internal sulfate attack, occur throughout the entire slab. For these distress types, partial-depth repairs are not a practical treatment because they do not address the entire extent of the deterioration and the patch itself will likely become debonded and quickly deteriorate.

A wide variety of materials are available for use in partial-depth patches. These include many rapid setting and high-early strength materials designed to reduce closure times. The selection of the most appropriate material depends on available curing time, ambient temperature, cost, and size of the repairs. Examples of materials include rapid setting cements, Duracal, Set 45, Five Star HP, and Percol FL. Bituminous patches are not recommended for repair of MRD on concrete pavements.

Full-Depth Repairs

Full-depth repairs are generally a better alternative than partial-depth repairs for addressing pavement deterioration caused by MRD. These repairs consist of the removal of isolated deteriorated areas through the entire depth of the slab and replacement with a high-quality material. Full-depth repairs are a widely used means of repairing localized deterioration at joints or cracks. As previously noted, MRDs are generally more severe along joints and cracks due to increased exposure to deleterious substances, which makes full-depth repairs an appropriate repair method.

In addition to replacing the deteriorated concrete, full-depth repairs also restore the load transfer at the joints. Consequently, full-depth repairs are an ideal method of repair for joints that have locked up and spalled due to corrosion of dowel bars. Likewise, wide cracks and punchouts where the reinforcing steel has corroded are also ideal candidates for full-depth repairs.

As with all repair methods, full-depth repairs should be viewed not as a solution to a durability problem but rather as a means to extend the life of the pavement. Because the problem is materials related, it cannot be completely remedied by replacing a portion of the pavement. However, full-depth repairs of badly deteriorated areas can improve the serviceability and buy additional life for the pavement. For pavements exhibiting ASR, full-depth repairs are only recommended to repair isolated areas that compromise the ride quality or safety of the users.

Because deterioration caused by MRD is often confined to transverse joints, full-depth repairs are an effective method of rehabilitation. The major shortcoming is that full-depth repairs create two joints where there originally was only one joint. In effect this adds one other potential problem area. Freeze-thaw deterioration of aggregate, for example, has been found to appear along the newly formed joints within 5 years. Treating the joint and possibly the adjacent concrete with a surface sealer can help prevent the recurrence, but even with preventive measures, full-depth repairs should be viewed only as a means to extend the life of the pavement.

The use of dowel bars is strongly recommended for full-depth repairs. Dowel bars provide better long-term performance by reducing vertical movements, rocking, and faulting. On high-volume roadways, the use of dowel bars is recommended on both sides of the patch. On lower volume facilities, tiebars may be used on the approach side of the patch with dowels used on the leave side of the patch. On CRCP, continuity of the reinforcing steel should be reestablished through the full-depth repair.

Slab Replacement

For deteriorated areas that are not isolated along joints or cracks or for large areas of deterioration, slab replacement may be a better alternative than full-depth repairs. Removal and replacement of multiple full-depth repairs becomes expensive, so it can become more cost effective to replace the entire slab. The problem is that MRDs generally occur throughout the entire project and are not likely to be limited to several slabs. Thus, it should be recognized that slab replacement does not completely address the durability problem (unless all slabs are replaced). Slab replacement can also be used in conjunction with other rehabilitation techniques to restore the pavement condition to an acceptable level.

Diamond Grinding

Diamond grinding uses closely spaced diamond saw blades to remove a thin layer of concrete from the pavement surface. This process corrects surface irregularities, provides a smooth riding surface, and improves the frictional characteristics of the pavement. For durability problems, the purpose of diamond grinding is simply to “buy some time” until more permanent rehabilitation can be conducted. As with the other rehabilitation methods, diamond grinding does not directly address the durability problem. Although diamond grinding will remove deteriorated areas from the pavement surface, a new layer of concrete will be exposed and will also deteriorate with time.

For diamond grinding to be feasible, the deterioration must be limited to the pavement surface. Diamond grinding should not be used in an attempt to repair more extensive deterioration. Surficial scaling and map cracking are examples of distress types that are good candidates for diamond grinding. Scaling can be attributed to the use of deicing chemicals or freeze-thaw deterioration of the cement paste. Map cracking is often associated with MRD that involves chemical reactions, such as ASR, ACR, and sulfate attack. Diamond grinding is also an effective method of restoring ride quality in conjunction with other repair methods such as partial-depth and full-depth repairs. If conducted, diamond grinding should be performed after any rehabilitation techniques (e.g., partial-depth or full-depth repairs) but before any surface treatment methods.

Overlay

Both AC and PCC overlays are common concrete pavement rehabilitation alternatives. Pavements with MRD are not good candidates for bonded PCC overlays whereas unbonded PCC overlays are a more feasible option. One problem with overlays is that they do not address a pavement’s durability problem. Distress in the underlying pavement will continue to deteriorate and will definitely affect the performance of the overlay. Additionally, the presence of the overlay may accelerate the rate of development as it traps more moisture in the existing pavement. As a result, the effect of MRD on the overlay must be taken into account during design. Means to account for existing MRD include preoverlay repairs of badly deteriorated areas and the use of a thicker overlay than would normally be used. Otherwise, it should be expected that the overlay will not provide the same performance or service life as an overlay placed over a pavement without MRD.

Given that an overlay covers the pavement surface and prevents direct exposure to the underlying pavement (reducing the depth of freezing temperatures), an overlay could also be considered a treatment method. The thought is that by covering the concrete with an AC overlay, the concrete will not experience the same number of freeze-thaw cycles. However, its effectiveness in this regard is limited. Temperature simulations in moderate climates found that a 150-mm overlay was not sufficient to prevent freezing in the underlying concrete pavement (Dempsey 1969). Table 1-6 presents the results of the study for various AC overlay thicknesses.

Although it is true that an AC overlay will reduce the number of freeze-thaw cycles, it can never completely eliminate freeze-thaw cycling in harsh climates. Previous studies have shown that, in order to stop the progression of freeze-thaw deterioration, freezing must be completely prevented; merely decreasing the number of freeze-thaw cycles with an overlay can actually accelerate the rate of deterioration (Janssen 1985; Janssen et al. 1986). Moreover, covering the concrete surface with an AC overlay can trap additional moisture in the pavement, which again can accelerate deterioration. Consequently, AC overlays are not recommended as a long-term solution on pavements with freeze-thaw deterioration of aggregate, but can be used as a temporary solution to buy some time until a more effective long-term solution is implemented.

Distress in the underlying pavement can also reflect through an AC overlay. Preventive measures (e.g., preoverlay repairs and reinforcing fabrics) can reduce or delay the occurrence of reflective cracking but are seldom totally effective at eliminating reflective cracking. An alternative is to fracture the existing pavement before placing an AC overlay, in which case the existing pavement acts as a base layer. This option has been successfully used to prevent reflective cracking [National Asphalt Paving Association (NAPA) 1991; FHWA 1998]. However, fracturing the existing pavement will reduce its structural capacity and thus it will require a thicker overlay.

An unbonded PCC overlay is another option (bonded overlays are not recommended on pavements showing MRD). Again, the overlay will not help prevent freeze-thaw cycling in the underlying pavement. However, an unbonded PCC overlay can be more effective than an AC overlay because its performance depends less on the condition of the underlying pavement. Providing uniform support is the most important consideration. The major shortcoming of unbonded PCC overlays is that they need to be thicker than other overlay types. This drawback translates to higher initial construction costs and potential problems with overhead clearances and grade differentials.

Table 1-6. Effect of overlay thickness on freeze-thaw cycling (Dempsey 1969).

Location	AC Overlay Thickness, mm	Number of Freeze-Thaw Cycles at Various PCC Depths		
		0 mm	50 mm	100 mm
Tulsa, OK	0	31.1	9.7	3.5
	50	11.2	2.5	1.2
	100	4.7	1.0	0.4
	150	1.7	0.5	0.3
Topeka, KS	0	52.6	17.5	7.1
	50	28.8	7.0	5.0
	100	11.5	5.5	4.0
	150	6.0	4.6	2.6
Lexington, KY	0	42.6	15.9	6.5
	50	23.6	6.5	3.7
	100	11.5	4.1	2.6
	150	4.8	3.1	2.5
Evansville, IN	0	40.7	—	—
	50	24.2	5.2	3.9
	100	10.9	2.8	2.0
	150	2.9	2.2	1.3
Dodge City, KS	0	54.2	16.9	5.9
	50	25.7	5.2	3.4
	100	10.7	3.7	2.9

Overlays should be used cautiously over pavements experiencing corrosion of embedded steel. If not repaired prior to overlaying, locked-up joints can lead to blowups in the concrete that will be just as damaging to the overlay. Temporary repairs to maintain serviceability until the pavement can be reconstructed may be a better alternative in this case.

Pavement Reconstruction

The most extreme rehabilitation alternative is total reconstruction of the pavement. This solution corrects an MRD problem and will prevent its recurrence if proper procedures are used to prevent MRD in the new pavement. Because durability problems are not often limited to isolated areas within the pavement, reconstruction is often the only practical long-term alternative. Of course, this alternative is most cost

effective on badly deteriorated concrete pavements, where mitigation methods are ineffective and repairs are too numerous and costly. Reconstruction may be the only option on pavements with MRDs that have progressed to high severity levels.

Pavement Recycling

Another form of reconstruction is pavement recycling. This process involves removal and crushing of the existing concrete for use as aggregate in the reconstructed pavement. Recycling offers several benefits over reconstruction, including reduced cost and conservation of materials. Recycling has been a viable option for decades but has only recently gained acceptance for pavements exhibiting durability problems. In such instances, adjustments to the mix design procedures must be made to prevent or limit the recurrence of durability problems in the recycled pavement. With these adjustments, recycling of concrete pavements exhibiting MRD can provide performance equivalent to that of conventional mixes (Wade et al. 1997).

Selection Process

The identification and analysis of MRD are critical steps in the selection of the most appropriate treatment or rehabilitation option. Not only is it important to identify the existing distress, but it is also important to understand the cause of the distress to prevent its recurrence. Once correctly identified, there are still multiple options from which to choose. Table 1-7 summarizes the potential treatment and rehabilitation alternatives for each MRD. These methods should only be viewed as possible alternatives for the particular MRD. Some methods will be more effective than others, as discussed previously. In addition, multiple MRD can appear concurrently, as well as with other distress types. The treatment and repair options must address all distress types.

Evaluation and selection of the proper repair material and technique are critical to ensuring adequate performance of the treatment or rehabilitation method over its expected performance life. The evaluation and selection process involves a large number of technical, economical, and practical factors. For example, the repair method must address the cause and prevention of MRD, as well as provide a certain level of serviceability over its performance life. Also, the proposed solution must be economically feasible in that it should provide a cost-effective solution in comparison to other potential alternatives. Finally, the proposed solution must be constructible using available materials, techniques, and equipment.

Selecting an appropriate treatment is a complex process. The selection of the most appropriate alternative must address the following questions:

- Is the distress affecting current traffic operations?
- What are the causes of distress?
- What are the consequences of not addressing the existing distress? How long before conditions deteriorate to the point where they become hazardous?
- Does the proposed treatment or repair option address the cause of the distress? Will the distress eventually recur?
- How will the application/construction of the proposed option affect traffic operations?
- What is the cost of the proposed treatment or repair option?
- What is the life expectancy of the treatment or repair? What is the remaining service life of the pavement?
- Does the proposed option coincide with future rehabilitation plans?

Table 1-7. Available treatment and rehabilitation options.

Type of MRD	Possible Treatment Methods	Possible Rehabilitation Methods
Freeze-Thaw Deterioration of Cement Paste	Surface sealing Sealing joints and cracks Retrofitted drainage Reconstruction	Partial-depth repairs Full-depth repairs Diamond grinding Slab replacement Overlay
Deicer Scaling/Deterioration	Surface sealing	Partial-depth repairs Full-depth repairs Diamond grinding Slab replacement Overlay Reconstruction
Freeze-Thaw Deterioration of Aggregate	Surface sealing Sealing joints and cracks Retrofitted drainage Crack filling (HMWM)	Full-depth repairs Reconstruction
Alkali-Silica Reactivity	Lithium salts Crack filling (HMWM) Sealing joints and cracks Retrofitted drainage	Full-depth repairs Reconstruction
Alkali-Carbonate Reactivity	Crack filling	Full-depth repairs Reconstruction
External Sulfate Attack	NaCl treatment Surface sealing Seal joints and cracks Retrofitted drainage	Full-depth repairs Overlay Reconstruction
Internal Sulfate Attack	NaCl treatment Seal joints and cracks Retrofitted drainage	Full-depth repairs Overlay Reconstruction
Corrosion of Embedded Steel	Surface sealing Sealing joints and cracks Retrofitted drainage	Full-depth repairs Overlay Reconstruction

This section discusses some of the key issues that must be balanced when comparing alternatives and selecting the optimal solution.

Distress Severity

The most appropriate treatment or rehabilitation method for pavements exhibiting MRD depends to a large extent on the severity of deterioration. As a general rule, treatment methods are most applicable to pavements exhibiting low-severity MRD. Pavements exhibiting high-severity MRD, on the other hand, are usually better candidates for rehabilitation methods. However, the selection will depend on a number of

other criteria, including the type of distress, the extent of the deterioration, overall pavement condition, traffic volumes, and anticipated life of the pavement.

Treatment methods are designed to stop or at least slow the rate of deterioration and are thus most effective for addressing MRD in its early stages of development. Treatment methods address the cause of deterioration. By applying treatment methods in the early stages of deterioration, the distress may be prevented (or at least delayed) from progressing to the point where it causes a serviceability problem.

Some treatment methods directly treat the cause of the distress by eliminating or altering an adverse reaction that creates the distress. Examples include the use of lithium salts to address ASR and the use of NaCl to address sulfate attack. Other methods, such as application with an HMWM, are designed to strengthen the pavement and prevent deterioration to the point where extensive repair is required. Still other methods are designed to hinder the exposure of the pavement to deleterious substances.

All of these methods must be applied in the early stages of deterioration. Once the deterioration has progressed to the point where it affects the serviceability or safety of the users, treatment methods will no longer be effective. For one, they will not address the serviceability or safety problem. Secondly, treating a pavement with severe distress is not cost effective because it will not eliminate the need for repairs.

Treatments that are designed to limit the exposure to moisture should be used with caution. Most methods are at best only partially effective at performing their intended function. For instance, resealing joints and cracks will not completely eliminate the infiltration of moisture (Hagen and Cochran 1995). Additionally, MRDs that require moisture typically initiate and deteriorate more rapidly at the bottom of the slab. By the time the distress is visible on the surface, the deterioration at the bottom is often too severe for treatment methods to be effective.

As MRDs progress to moderate-severity levels, treatment methods become less effective. This is especially true for distresses that initiate at the bottom of the slab, such as freeze-thaw deterioration of aggregate and external sulfate attack. The one exception is the use of crack fillers such as HMWM. To be effective, crack fillers must be able to penetrate the pavement surface. Cracks that are slightly open provide easier access for the filler material to penetrate.

Often, a combination of treatment and rehabilitation methods will be the best alternative for moderate-severity MRD. Rehabilitation methods should be employed to address deteriorated areas and restore serviceability. Treatment methods can then be used to prevent further progression of the MRD. For example, salt scaling can first be addressed through diamond grinding, followed by a surface sealer to prevent continued deterioration.

Rehabilitation methods are generally the only available alternative for MRDs that have progressed to high-severity levels. At this point, the pavement is too deteriorated, and treatment methods no longer offer an effective means of addressing the problem. Badly deteriorated areas must be removed and replaced with a suitable material in order to restore serviceability. Full-depth repairs are a common technique used to address MRDs that are limited to transverse joints and cracks. As the deterioration becomes more extensive, major rehabilitation techniques such as overlays or reconstruction may be the only viable alternative.

Distress Quantity

Another important consideration when selecting the most appropriate alternative is the amount of distress, both in terms of the extent and depth of deterioration. The distress mechanism influences the extent and depth of deterioration and must be considered when determining the treatment or rehabilitation method.

Distresses that are concentrated at joints and cracks are good candidates for localized techniques such as partial-depth and full-depth repairs. Alternatively, more widespread distresses require more extensive techniques such as a chemical treatment, diamond grinding, or an overlay. They cannot be addressed cost effectively with localized repair methods.

Examples of MRD that are candidates for localized rehabilitation techniques include freeze-thaw deterioration of aggregate and corrosion of embedded steel. Both distresses are concentrated at joints and cracks where they have greater exposure to moisture and deleterious chemicals. These distresses generally do not occur or are much less severe in other areas of the slab. Consequently, full-depth repairs are the most common method used to address these distresses. Salt scaling can also occur in localized areas such as depressions where water puddles for extended periods and poorly finished areas. Partial-depth repairs can be used to address these localized areas of deterioration if they are restricted to the upper third of the slab.

More extensive MRDs include ASR, ACR, and sulfate attack. These distresses involve a reaction between constituent materials within the concrete. Although they can be more severe near joints and cracks, they generally occur throughout the entire slab. More extensive treatments must therefore be employed to address the deterioration. Chemical treatments and surface sealers are examples of two treatment methods that address widespread deterioration. Widespread rehabilitation methods include diamond grinding, overlays, and reconstruction.

Along with the extent of deterioration, the depth of deterioration is also a determining factor in selecting the appropriate method. The extent of deterioration at the bottom of the slab is just as important as the surface condition when selecting the most appropriate treatment method. Many MRDs initiate at the bottom of the slab and progress upward. By the time the distress is visible at the surface, it can be quite extensive at the bottom of the slab. Some treatment and rehabilitation methods only address deterioration at the slab surface and will not be effective for deterioration that extends through the depth of slab.

Coring at representative locations throughout the pavement is recommended to examine the extent of deterioration through the depth of the slab. At least two cores should be taken along transverse joints, longitudinal joints, cracks, and midslab locations. This preliminary investigation is useful for planning purposes. Nonetheless, if the deterioration is found to be more extensive during the construction phase, adjustments must be made. For instance, a partial-depth repair requires that the deteriorated area be completely removed. If the deterioration extends beyond the upper third of the slab thickness, complete removal and a full-depth repair are called for. Failure to remove the extent of deterioration will result in premature failure of the repair.

Overall Pavement Condition

The selection of the most appropriate treatment or rehabilitation method must consider the overall pavement condition and not just the MRD itself. The extent and severity of other distress types are equally important and must also be addressed. The most cost-effective solution is one that addresses all distress types simultaneously. In some cases, this may require a combination of two or more treatment or rehabilitation methods.

Just because a method is the most effective treatment or rehabilitation option for a particular MRD does not mean it is the most appropriate alternative for the pavement. For instance, the most effective method for a pavement exhibiting freeze-thaw deterioration of aggregate that is confined to the joints might be treatment with a HMWM. However, if the joints are also faulted or locked-up, full-depth repair will likely be a more cost-effective option because it addresses both problems.

Similarly, the selected alternative must fit in with the future plans of the roadway. It would be unwise and wasteful to apply a treatment to address ASR when the pavement is expected to receive major rehabilitation in a few years. Along the same lines, the rate of deterioration should also be considered.

For example, the rate of expansion caused by DEF follows an S-shaped curve (Thaulow et al. 1996a). The selection of the treatment or repair option should thus consider the point in the reaction process. Expansion due to DEF typically occurs between 2 and 6 years after construction.

Predicted Performance

The expected performance of an MRD treatment alternative is a key issue in the selection process. Not only is it important to ensure that the method will effectively address the problem, but it is also important to know how long the pavement will provide acceptable serviceability once the method is applied. The performance life of the method, along with the cost of the method, are the major considerations in the conduct of a life cycle cost analysis.

The past performance of treatment and rehabilitation methods for each MRD has been presented. Some of the MRDs are extremely complex, so there still exist some unknowns as to the exact mechanisms and causes of the distress. Likewise, many of the available treatments to address MRD are still in the experimental stages. Consequently, little or no field performance data are available for many of the available methods, especially for some of the treatment alternatives. These factors add to the difficulty of predicting the increased performance life through application of the treatment or rehabilitation method. Agencies may need to experiment with treatments on a small scale to gauge their effectiveness.

The predicted performance (life) of both the existing pavement and the treatment or rehabilitation method must be considered together. It would be foolish to apply a treatment or rehabilitation that will last 10 years when the pavement is expected to fail by other means within 5 years. Frequently the best solution is to apply lower cost methods as a temporary fix to the problem, designed to maintain rideability while more permanent solutions are planned.

Life Cycle Cost Analysis

A life cycle cost analysis procedure provides a valuable tool for selecting the alternative that will provide the required performance at the lowest cost (i.e., the most cost-effective alternative). The procedure needs to consider all costs associated with a given alternative, including initial application costs, future maintenance and rehabilitation costs, user costs, and salvage value. Many of these costs are difficult to predict with any accuracy.

For years, a deterministic approach to predicting life cycle costs has been employed. This method uses discrete inputs to predict a discrete cost. Numerous publications are available regarding this approach (for example, Peterson 1985). More recently, there has been increased interest in a probabilistic approach. This approach considers variations in the inputs to compute a range of results and the probability of occurrence. The FHWA has recently developed a Technical Bulletin that provides guidelines for using this approach in pavement design (FHWA 1997).

A life cycle cost analysis should be conducted on each feasible alternative. However, the life cycle cost analysis should not be used as the ultimate decision-maker. Although it is an extremely valuable tool, life cycle costs represent only one of many factors that need to be considered. Factors such as the reliability of the method and overall project planning are equally important.

Summary of Background on Treatment and Rehabilitation Methods

Pavements inflicted with MRD can be addressed through either treatment or rehabilitation methods. Treatment methods focus on eliminating or reducing rate of deterioration and are generally most appropriate on pavements exhibiting low-severity MRD. Rehabilitation methods, on the other hand, involve removal and repair of the distressed area and are most appropriate for addressing high-severity

MRD. Moderate-severity MRD must further consider the extent of deterioration or may involve a combination of treatment and rehabilitation methods.

This background presented the state-of-the-practice of treatment and rehabilitation methods designed to address MRD. Currently, there are no clear-cut guidelines to address a particular MRD. Today's sophisticated equipment has led to sweeping changes in the investigation and identification of MRD. Researchers continue to developing a greater understanding of the distress mechanisms. Although this is an important step, the technology for treating MRD has not yet advanced to the same level. Several treatment methods have been developed and are currently undergoing laboratory and experimental field evaluations. For the most part, however, the treatments have not yet received widespread use in the field.

Approach for Treatment and Rehabilitation of Concrete Pavements Affected by MRD

This section presents a brief review on the approach taken for the selection of the most appropriate treatment or rehabilitation options to address MRD in concrete pavements. The approach is described in detail in the guideline presented in Volume 2 of this Final Report. The process involves consideration of a large number of technical, economical, and practical factors, as well as coordination with the overall pavement condition and future rehabilitation plans. For example, serviceable concrete repairs can result only if the MRD is correctly identified, the proper materials and methods selected, and good construction practices followed. The proposed solution must also be economically feasible in that it should provide a cost-effective solution in comparison to other potential alternatives. Finally, the proposed solution must be practical, being able to be performed using available materials, techniques, and equipment.

Figure 1-9 presents the recommended flowchart for selecting the preferred treatment or rehabilitation option. Although the overall process is the same regardless of the type of MRD exhibited, portions of the guidelines — such as the selection of feasible alternatives and repair materials — are further broken out by MRD type. The overall process also considers means to address pavements exhibiting multiple distress types, including distresses that are not caused by durability problems.

The first step is to identify the extent and severity of MRD on the existing pavement as described in the first two guidelines in this series. The identification and analysis of MRD is a critical step in the selection of the most appropriate treatment or rehabilitation option. Not only is it important to identify the existing distress, but it is also important to understand the cause of the distress to prevent its recurrence. Assessing the rate of deterioration is also important to determine the progression in the deterioration process.

Once the type, extent, and severity of the MRD are characterized, the next step involves the selection of feasible alternatives, which differ depending on the type(s) of MRD identified. Although the overall process is the same, the feasible alternatives will be different for each MRD. These differences are addressed through corresponding tables that are referenced for each MRD.

The final step is to select the most appropriate option from the list of feasible alternatives. This process involves an evaluation of many considerations, including potential constraints, future rehabilitation activities, expected performance, and life cycle costs.

Recommended Techniques and Materials for Treatment and Rehabilitation

The treatment and rehabilitation techniques discussed in the previous section were adopted in the guideline. The methods to address existing MRD in concrete pavements are divided into treatment methods (those designed to prevent further development of the distress or to reduce its rate of deterioration) and rehabilitation methods (those designed to remove deteriorated areas and to maintain or restore pavement serviceability). As described previously, the available treatment methods include chemical treatments, joint and crack sealing, crack filling, surface sealing, and retrofitted drainage. Drying

and restraint are two other treatment methods that were described, but are not practical for pavement applications, and are thus not considered in the treatment options. Rehabilitation methods include partial- and full-depth repairs, slab replacement, diamond grinding, reconstruction, and recycling.

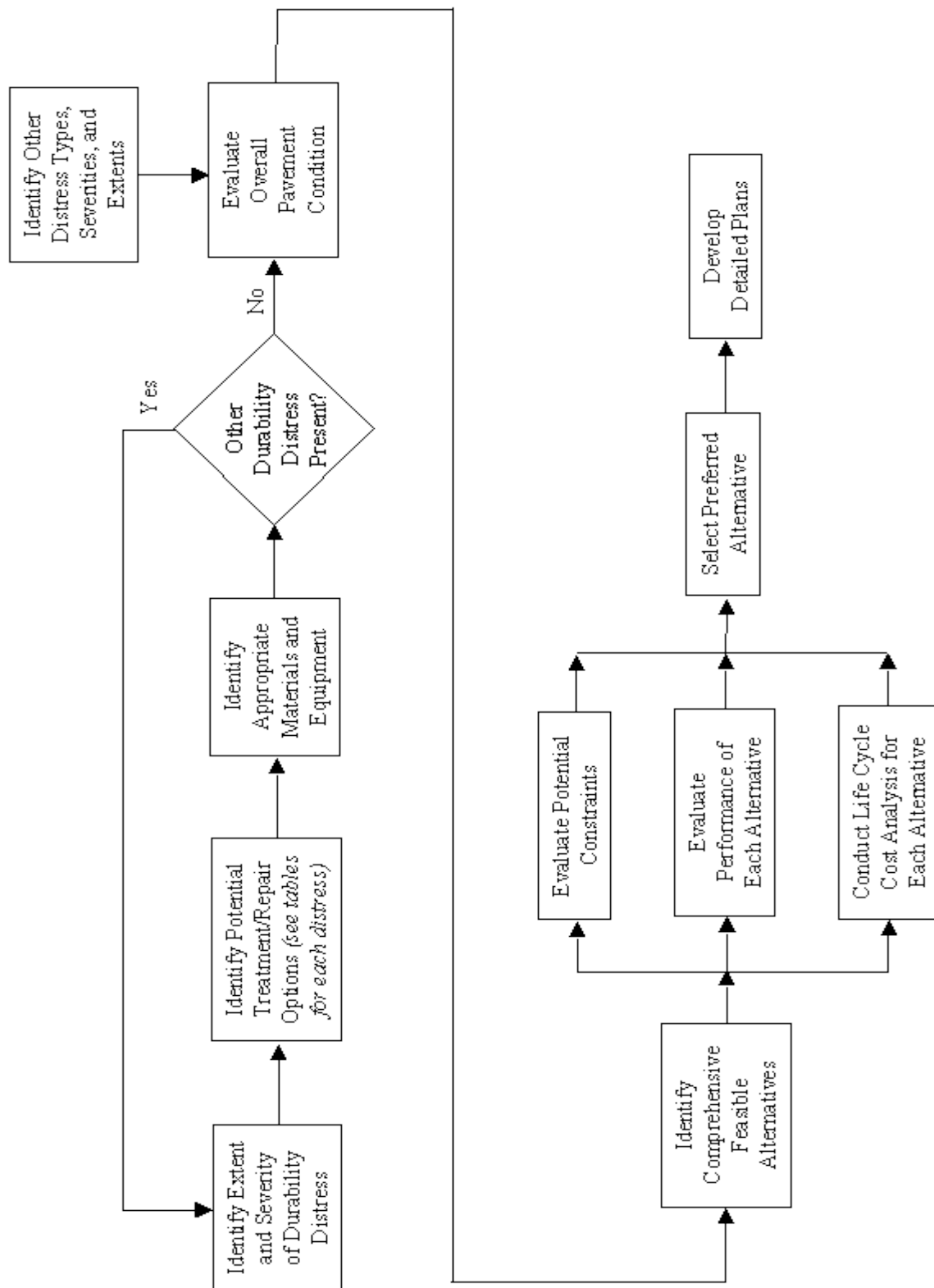


Figure 1-9: Flowchart. Flowchart for selecting preferred treatment and rehabilitation options.

Feasibility of Available Techniques

A "feasible" alternative must address both the current condition and future performance of the pavement. The identification of feasible alternatives varies significantly depending on the type(s) of MRD occurring. In the guideline, a recommended approach for selecting feasible alternatives is provided in tabular form for each particular MRD type. For each MRD type, the following issues are discussed:

- Available treatment and rehabilitation methods.
- Effectiveness of each method.
- Feasible alternatives based on the extent and severity of distress.
- Past performance and predicted life of each alternative.

It is noted that it is not uncommon that more than one type of MRD may be at work within a given pavement. Fortunately, many of the treatment and rehabilitation alternatives are equally effective for a number of different MRD types. But it is important to consider the feasibility of the alternatives based on all the mechanisms at work if multiple mechanisms are observed.

Selection of Preferred Alternative

The final series of steps involves the selection of the most appropriate alternative to address the overall needs of the pavement. Selecting the most appropriate method and technique is a complex process involving a large number of technical, economical, and practical considerations. The previous discussions for each distress focus on the effectiveness of the methods. However, many other considerations must be addressed in the final selection process. The selection of the most appropriate alternative must address the following questions:

- What are the causes of distress?
- What are the consequences of not addressing the existing distress? How long before conditions deteriorate to the point where they become hazardous?
- Does the proposed treatment or repair option address the cause of the distress? Will the distress eventually occur again?
- How will the application/construction of the proposed option affect traffic operations?
- What is the cost of the proposed treatment or repair option?
- What is the life expectancy of the treatment or repair? What is the remaining service life of the pavement?
- Does the proposed option coincide with future rehabilitation plans?

Some of these issues have already been addressed. This section discusses the remaining issues that must be balanced when comparing alternatives and selecting the optimal solution.

Overall Pavement Condition

The selection of the most appropriate treatment or rehabilitation method must consider the overall pavement condition and not just the MRD itself. The extent and severity of other distress types are equally important and must also be addressed. The most cost-effective solution is one that addresses all distress types simultaneously. In some cases, this may require a combination of two or more treatment or rehabilitation methods.

Possible Constraints

Certain constraints may limit the feasibility of one or more techniques and should be considered in the selection process. At times, these factors take precedence over all other considerations, including the

effectiveness of available methods. Such overriding factors may be the result of traffic, climate, materials, or construction considerations. Examples of possible constraints include the following:

- Limited project funding.
- Traffic control requirements.
- Future maintenance and rehabilitation.
- Overhead clearances.
- Agency policies.
- Available materials and equipment.
- Contractor expertise and manpower.

Predicted Performance

The expected performance of an alternative is a key issue in the selection process. Not only is it important to ensure that the method will effectively address the problem, but it is also important to know how long the pavement will provide acceptable serviceability once the method is applied. The performance life of the method (along with the cost of the method) are the major considerations in the conduct of a life cycle cost analysis.

The predicted performance (life) of both the existing pavement and the treatment or rehabilitation method must be considered together. For example, it is ineffective to apply a treatment or rehabilitation that will last 10 years when the pavement is expected to fail by other means within 5 years. Often times, the best solution is to apply lower cost methods designed to maintain rideability as a temporary fix to the problem while more permanent techniques are planned.

Life Cycle Cost Analysis

A life cycle cost analysis procedure is a valuable tool for selecting the alternative that will provide the required performance at the lowest cost (i.e., the most cost-effective alternative). The procedure needs to consider all costs associated with a given alternative, including initial application costs, future maintenance and rehabilitation costs, user costs, and salvage value. Many of these costs are difficult to predict with any accuracy.

For years, a deterministic approach to predicting life cycle costs has been employed. This method uses discrete inputs to predict discrete cost. Numerous publications are available regarding this approach (for example, Peterson 1985; Van Wijk 1985). More recently, there has been increased interest in a probabilistic approach. This approach considers variations in the inputs to compute a range of results and the probability of occurrence. The FHWA has published a report that provides guidelines for using this approach in pavement design (Walls and Smith 1998).

A life cycle cost analysis should be conducted on each feasible alternative. However, the life cycle cost analysis should not be used as the ultimate decision-maker. Although it is an extremely valuable tool, it represents only one of many factors that need to be considered. Factors such as the reliability of the method and overall project planning are also important.

Construction and Maintenance Considerations

The methods for addressing MRD often require special considerations for construction and maintenance. The general construction and maintenance recommendations are contained in the guideline for each of the available, practical methods. The recommendations focus on differences or special considerations that are required for pavements with MRD as compared to traditional techniques.

Summary of Recommended Treatment and Rehabilitation Methods

The guideline contained in Volume 2 of this final report presents information on selecting the most appropriate treatment or rehabilitation option to address MRD on concrete pavements. Treatment methods focus on eliminating or reducing rate of deterioration and are most appropriate on pavements exhibiting low-severity MRD. Rehabilitation methods, on the other hand, involve removal and repair of the distressed area and are most appropriate for addressing high-severity MRD. Specific guidelines for each distress type are provided.

A variety of treatment methods are available to address MRD. However, many of the methods are still being tested in the laboratory and have not yet received widespread use in the field. Nonetheless, this guideline presents the most recent information on the effectiveness of the methods to address MRD.

Background on the Prevention of MRD in Concrete Pavements

The previous sections in this report have reviewed literature focused on the identification of MRD in concrete pavements and the treatment and rehabilitation alternatives that are available to address the observed deficiencies. This section of the report presents methods to prevent the occurrence of MRD in new pavements. This is considered to be of utmost importance as the available treatment and rehabilitation options are limited once MRD occurs.

In order to construct durable concrete pavements, it is necessary to approach the selection of residual materials, mixture design, and construction from a holistic point of view. The literature contains numerous examples describing how two or three MRD mechanisms appear to be at work simultaneously, making it nearly impossible to separate the actual "cause" of distress from the opportunistic distress that became manifest only after degradation had already begun. By adopting a more holistic approach in which the entire concrete pavement is viewed as a system, more durable concrete will result.

Mehta (1997) describes this concept when he states that "current theories on the mechanisms responsible for deterioration of concrete due to various causes are based on a reductionistic approach to science. . . " that tries to understand a complex system by reducing it to parts and then considering only one part of the problem at a time. As a result, current material specifications and test methods are focused only on a series of single attributes, failing to consider the system as a whole. Metha (1997) advocates a holistic approach in which loss of watertightness is the primary concrete deterioration mechanism, which in turn results in loss of strength and stiffness, and MRD. His holistic model for reinforced concrete structures is presented in figure 1-10. A similar model could be developed for concrete pavements.

In a similar vein, Leek et al. (1995) provide recommendations for general chemical attack resistance. They recommend that minimizing voids and cracks, ensuring a good bond between aggregate and cementitious paste, minimizing porosity of the paste, and minimizing the paste fraction of the concrete can all improve resistance to chemical attack through decreased permeability.

This section of the report presents the background information on prevention of specific types of MRD and a brief summary of the recommendation espoused in the guidelines presented in Volume 2 of this Final Report for the prevention of MRD in concrete pavements.

It is noted that a vast body of knowledge already exists concerning the mix design and construction of durable concrete. This section is not written to replace that body of knowledge, but instead to highlight the key information that is directly relevant to the topic of concrete pavement durability. Information contained in such accepted works as *The Design and Control of Concrete Mixtures [Portland Cement Association (PCA) 1992]*, *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass*

Concrete(ACI 1991), the *Guide to Durable Concrete*(ACI 1992a), and the *Durability of Concrete*(TRB 1999) still form the basis for concrete mixture design, construction, and durability.

The background information on the prevention of MRD in concrete pavements is first presented as preventive strategies for addressing specific types of MRD, and then focuses on an overall approach to materials selection, mixture design, testing, and construction considerations that will result in more durable concrete pavements.

Preventive Strategies for Controlling Specific Types of MRD

This section of the report focuses on strategies for preventing the development of specific MRD types. It is noted that the overall intent is to create watertight, relatively defect-free concrete of the highest quality and durability. This section also presents techniques for mitigation that can be applied to address potential problems with constituent materials noted during materials screening.

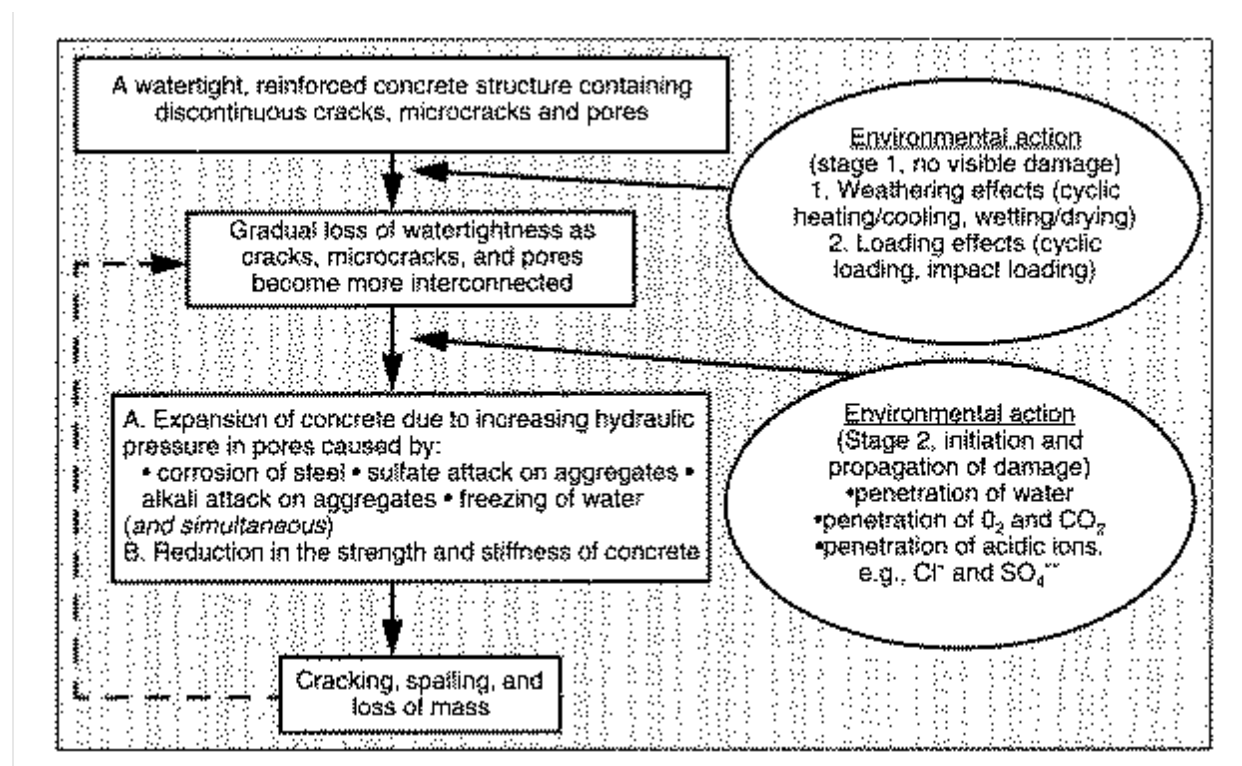


Figure 1-10. A holistic model of concrete deterioration from environmental effects (Mehta 1997).

Preventive Strategies for Controlling Paste Freeze-Thaw Deterioration

Perhaps the most profound discovery concerning the resistance of concrete to freeze-thaw damage was that the entrainment of microscopic air voids could vastly improve the resistance of the paste phase to such damage. As early as the 1930's, deliberate efforts were made to take advantage of this beneficial use of entrained air. Since then, it is standard practice to entrain air voids of the proper size and spacing into the concrete matrix to protect it against paste freeze-thaw damage.

It has been speculated that concrete produced with a w/c below about 0.25 will not require an entrained air system for freeze-thaw durability. Clearly, if the capillary pore system could be entirely eliminated at a low enough w/c , no freezable water would exist in the microstructure. In practice, it is unreasonable to expect to eliminate all capillary pores. Philleo (1983) notes that the combination of self-desiccation, drops in internal humidity with loss of available water, and the statistical non-uniformity of the developing capillary pore system will likely result in some capillary porosity even in very low w/c concrete. Thus, some form of entrained air system is still likely to be needed even for very low w/c concrete.

Air entraining admixtures are specified and tested under ASTM C 260 and C 233. Added to properly proportioned and mixed concrete at established dosage rates, an adequate air void system should be produced. Individual air-entraining admixtures tend to produce characteristic void size distributions regardless of the total volume of entrained air. In general, lower w/c , finer ground cements, finely divided mineral admixtures, and a high proportion of fines in the fine aggregate will typically result in less air entrained in a concrete mixture (Mindess and Young 1981). Because the entrainment of air is the direct result of the mixing action of the mixer, the mixing conditions have a significant impact on the resulting void system. The amount of entrained air initially increases with mixing, but then gradually declines with prolonged mixing.

The air content of fresh concrete can be determined using ASTM C 173 or C 231. Although air content is the parameter typically measured during construction, it alone does not ensure that the air void system in the hardened paste is adequate. Loss of some air during slipform paving is common. Also, the overall air content measurement does not distinguish between entrapped air and entrained air. Recommended air contents for various exposure conditions and maximum aggregate sizes are presented in table 1-8.

In addition to its direct effect on freeze-thaw durability, entrained air affects other properties of fresh and hardened concrete. Entrained air typically improves the workability of fresh concrete and tends to reduce bleeding and segregation. At a constant w/c , the addition of internal air voids will have a detrimental impact on the strength of hardened concrete. For moderate strength concrete, each 1 percent increase in air content tends to reduce compressive strength by about 5 percent. This reduction in strength can be partially offset by a reduction in the w/c made possible by the increased workability of air-entrained concrete. For higher strength concrete, a similar reduction in strength occurs with the addition of air, but it has been shown that with the use of high-range water-reducing admixtures, strength can be maintained by a modest reduction in w/c . When the reduced w/c that is made possible by entrained air is utilized, the sulfate resistance of concrete is also improved because of the resulting reduction in permeability.

If difficulties in obtaining adequate air entrainment have been encountered, the air void system in hardened concrete should be assessed microscopically as part of the mix design and during construction using procedures described in ASTM C 457. The use of automated air void analysis will significantly reduce the time and labor needed to conduct this evaluation. It is currently the only accepted means to determine if the air-void system is adequate to protect the paste from freeze-thaw deterioration.

In situations where direct laboratory testing is desired to establish freeze-thaw performance, ASTM C 666 (Method A) *Resistance of Concrete to Rapid Freezing and Thawing*, is the most widely used standardized method for accelerated freeze-thaw testing. The method subjects concrete test prisms to 300 cycles of freezing and thawing. Two different procedures are provided in the method: Procedure A, in which both the freezing and thawing of the test specimens occur in water, and Procedure B, in which the freezing of the test specimens occurs in air while the subsequent thawing is in water. ASTM C 666 was created in 1971 by consolidating two different procedures, ASTM C 290 for Procedure A and ASTM C 291 for Procedure B, that had been in use since 1952 (Newlon and Mitchell 1994). In response to criticisms of both these procedures, including concern about possible confining pressures on the specimens created in Procedure A and possible drying of the specimens during freezing in Procedure B, a third procedure has been proposed (Janssen and Snyder 1994). The proposed alternative (termed Procedure C) modifies Procedure B by wrapping the test specimens with absorbent cloth to help keep them wet during freezing. Reduced variability in test results was reported using the proposed Procedure C. Current recommendations, however, are to use Method A (TRB 1999).

Table 1-8. Recommended air contents for freeze-thaw distress resistant concrete (ACI 1992a).

Nominal Maximum Aggregate Size, mm (in)	Average Air Content, Percent ¹	
	Moderate Exposure ²	Severe Exposure ³
9.5 (3/8)	6	7.5
12.5 (1/2)	5.5	7
19 (3/4)	5	6
25 (1)	5	6
37.5 (1-1/2)	4.5 ⁴	5.5 ⁴
75 (3)	3.5 ⁴	4.5 ⁴
150 (6)	3	4

¹A reasonable tolerance for air content in field construction is ± 1.5 percent.

²Outdoor exposure in a cold climate where the concrete will be only occasionally exposed to moisture prior to freezing, and where no deicing salts will be used. Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil.

³Outdoor exposure in a cold climate where the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, and water tanks.

⁴These air contents apply to the whole as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 37.5 mm (1-1/2 in) is removed by handpicking or sieving and the air content is determined on the minus 37.5 mm (1-1/2 in) fraction of the mixture. (The field tolerance applies to this value.) From this the air content of the whole mixture is computed.

If specimens fail ASTM C 666, petrographic analysis using ASTM C 856 can be used to determine which component of the concrete mixture is responsible so that modifications can be made. This test is more time consuming and labor intensive than ASTM C 457, and therefore is not recommended to test paste freeze-thaw deterioration alone.

Preventive Strategies for Controlling Deicer Scaling/Deterioration

In general, deicer scaling is not a concern for properly constructed, high quality PCC. But even if the concrete is properly constructed and cured, deicers may damage concrete with poor mixture characteristics, or concrete that was improperly finished. Pigeon (1994) comments on this by stating that "air entrainment improves to a very large degree the resistance to deicer salt scaling...and as could be expected, scaling decreases with water-cement ratio, and finishing and curing operations are particularly important." As indicated, the two primary mix design considerations for producing deicer scaling/deterioration resistant concrete are relatively high cement content (with corresponding low w/c) and air entrainment.

Air content is an important consideration when trying to prevent deicer scaling. Concrete that has adequate air entrainment for protection against freeze-thaw damage may be susceptible to the development of salt scaling. Collepardi et al. (1994) conclude that air contents recommended by ACI for "severe exposure" should be followed if the concrete is to be exposed to calcium chloride deicers, finding that concrete with air contents meeting ACI "moderate exposure" requirements suffered severe degradation in laboratory testing even without being subjected to freeze-thaw action. The ACI recommendations are presented in table 1-8 (ACI 1992a).

One controversial mix design consideration is cement content. Some studies have reported that to produce deicer scaling/deterioration resistant concrete, a relatively high cement content (with corresponding low w/c) should be used. A recent ACPA (1992) publication states that salt scaling can be prevented if a minimum cement content of 335 kg/m^3 and a maximum w/c of 0.45 are used. In earlier work, Woods (1968) recommended that a minimum cement content of 370 kg/m^3 be used to prevent scaling. Marchand et al. (1994), in their review of deicer salt scaling, cites a number of studies that show as w/c increases, so does scaling. They found that cement content is not the critical issue, but instead a low w/c with adequate workability to facilitate consolidation is needed. This point is reinforced in a recent publication that states that the practice of specifying a minimum cement content to ensure durability is "obsolete" (TRB 1999).

In addition to the use of entrained air and low w/c , the use of fly ash has been proposed to enhance deicer scaling/deterioration resistance. The PCA (1992) reports that air-entrained concrete containing fly ash has similar freeze-thaw durability to concrete made with portland cement as the sole binder as long as the same compressive strength, air void system, and curing are obtained. The results of a study conducted by Malhotra et al. (1991) agree with those by Malek and Roy (1988) in which it was found that the incorporation of fly ash (two ASTM Type F and one Type C) appreciably reduced the permeability of concrete to chloride ions. This study was based on measuring chloride ion permeability using AASHTO T 277 and not on a measure of scaling resistance. Bilodeau et al. (1991) found that concrete containing up to 30 percent ASTM Type F fly ash generally performed well under the combined effect of freezing and thawing in the presence of salt deicer, although performance of the fly ash concrete was more variable. Although extended periods of moist-curing or drying periods did not seem to significantly affect performance, the use of membrane curing had a decided benefit, particularly for fly ash concrete. In a paper published a year later, Bilodeau and Malhotra (1992) reported on the properties of concrete containing ASTM Type F fly ash as 58 percent of the total cementitious materials content. He noted that, in addition to acceptable mechanical properties, high-volume fly ash concrete had excellent resistance to chloride ion penetration as compared to plain concrete mixes. He mentioned that further testing is needed to demonstrate scaling resistance.

An experiment conducted by Byfors (1987) examined the influence of fly ash and silica fume addition on chloride ion penetration and pore solution alkalinity. It is reported that the addition of either fly ash or silica fume considerably reduces the rate of chloride ion diffusion. Malhotra et al. (1991) cite a number of studies that indicate the incorporation of low calcium fly ash (ASTM Type F) in concrete significantly reduces chloride ion penetration.

In a large laboratory study, Gebler and Klieger (1986) examined the durability of concrete made with both ASTM Type C and Type F fly ash. Both the chloride ion penetration and deicer scaling resistance were measured. It was concluded that fly ash concrete was just as resistant to chloride ion penetration as conventional concrete with similar w/cm . But it was also found that conventional concrete was more scaling resistant than fly ash concrete, regardless of the type of fly ash used.

Marchand et al. (1994), in a review of salt deicer scaling, report that many studies are contradictory concerning the scaling resistance of fly ash concrete. They found that many laboratory studies indicate poor scaling resistance of fly ash concrete, yet field studies show adequate performance. They attribute most of these differences to the variable nature of fly ash and to its slow rate of hydration, concluding that the maximum recommended amount of fly ash should be limited to 30 percent of the total mass of cementitious material.

Detwiler et al. (1994) found that either a 5 percent addition of silica fume or 30 percent addition of slag led to a far greater reduction in chloride ion penetration than did reducing the w/c from 0.5 to 0.4. They also noted that increasing the curing temperature over a range from 23°C to 70°C creates large differences in performance that indicates "the importance of controlling the curing process if durability is to be achieved." Their findings seem inconsistent with those of Stark and Ludwig (1997), but this may be explained by Malhotra's findings in which concrete containing fly ash actually has lower chloride

permeability but still had poor scaling resistance (Malhotra 1991). Obviously, more research needs to be conducted to determine if fly ash and/or GGBFS can be used to improve scaling resistance.

Whiting and Schmitt (1989) developed a model that related surface scaling to various paste parameters for concrete produced using high range water reducers. The published model is reported below:

$$SC = -4.187 + 7.313\bar{L} + 8.939 w/c + 0.372VR \quad (1-7)$$

where:

SC= The scaling code (from 1 to 5).

$\bar{L}$ = The Power's spacing factor, mm.

w/c=The water-cement ratio.

VR= The void removal parameter (1 to 5).

This model predicts deicer scaling according to a scaling code (SC), with 1 meaning no scaling observed and 5 being heavy scaling. The model predicts that SC is reduced by minimizing the Power's spacing factor ($\bar{L}$), by decreasing the w/c, and by decreasing the void removal parameter, VR. The void removal parameter is a factor that considers the proportionate variability in the amount of voids that exist in the near surface zone of the concrete compared to the main concrete mass. It ranges from 1 to 5, with a 1 indicating that no variation in the void structure is apparent and a 5 indicating complete removal of voids from the near surface zone. Thus, two of the three factors contributing to scaling are mix related ($\bar{L}$ and w/c), while the third factor is controlled by construction.

Construction and curing considerations include good finishing practices, adequate curing, and a drying period prior to deicer application. Deicer scaling is more likely to occur in concrete that has been overvibrated or improperly finished, actions that create a weak layer of paste or mortar either at or just below the concrete surface (Mindess and Young 1981). As discussed, this layer consists primarily of paste and is subjected to high hydraulic and osmotic pressures as well as thermal shock upon the application of chemical deicers.

The ACPA (1992) states that the prevention of salt scaling can be accomplished by "providing adequate curing, and providing an absolute minimum of 30 days of concrete 'drying' before applying deicing chemicals." The term "adequate curing" is a matter of some debate. Woods (1968) recommends that periods of moist curing be related to cement type and curing temperature as shown in table 1-9. Interestingly, in Woods' text, the word "moist" is not associated with curing since it is assumed that all curing is moist curing. It is also worth noting that less than 7 days moist curing was not even considered an acceptable alternative.

Table 1-9. Minimum moist curing times for scaling resistance (Woods 1968).

Cement Type	Percent CaCl ₂ (accelerator)	Minimum moist curing period, days		
		at 23°C	at +4°C	at -4°C
I	0	7	15	over 60
	2	7	7	30
II	0	7	12	35
	2	7	7	28
III	0	7	7	24

In work conducted by Beaupré and cited by Pigeon and Plateau (1995), it was confirmed that moist curing is very beneficial in minimizing deicer scaling, as determined using the ASTM standard test C 672, with 14 days moist curing being significantly better than 2 days. An interesting finding of this study, and one confirmed by other researchers, is that a 7-day curing period with curing compounds is superior to moist curing. The exact reason for this is unknown, as it was shown that it is not a result of lower chloride ion penetration. Pigeon and Plateau (1995) speculate that it might be due to slowing down the drying process, with a concomitant reduction in surface damage due to drying. How an application of a curing compound can reduce drying to below that which occurs under moist curing is unexplained, as are the uncertainties as to whether this benefit transfers from the laboratory to the field.

Gunter et al. (1987) report that the length of wet curing affects the resulting pore structure, with extended wet curing resulting in a finer, more uniform pore size distribution. This has a positive effect on durability. Interestingly, he found that long periods of curing in water can actually critically saturate voids, decreasing durability to freeze-thaw. Overall, he concludes that only a short duration of curing is required for conventional air-entrained concrete to produce resistance to freezing and thawing and deicer scaling.

Research reported by Tsukinaga et al. (1995) also has some bearing on concrete curing. They studied the effectiveness of permeable sheets used during construction to improve concrete properties. Permeable sheets are relatively new to concrete construction, typically being placed between the formwork and concrete in vertical construction. The permeable sheet allows bleed water to drain down and air to escape upward, densifying the concrete surface. This densification of the surface has positive effects in reducing "bug holes," decreasing the w/c of the surface, increasing strength, reducing permeability, improving freeze-thaw resistance, and decreasing chloride ion penetration. It is uncertain if similar technology would be practical for horizontal flat work or pavement construction. But it is interesting how an improvement of the concrete surface characteristics through modified construction practices can have a profound effect on factors related to durability.

Once a mixture is designed, it is important to determine its deicer scaling resistance. Two test methods are commonly used to investigate the scaling resistance of concrete. The most commonly used test is ASTM C 672 (Mindess and Young 1981). In this test, a CaCl₂ solution is ponded on the surface of rectangular concrete specimens that are then subjected to freeze-thaw cycling. The specimens are placed in a freezer (-17.8°C) for 16 to 18 hours and then manually removed to a thawing environment for 6 to 8 hours. A surface layer of the water/salt solution is maintained at all times. A visual inspection is made at 5, 10, 15, 20, 25, and 50 cycles and the concrete is rated on a scale of 0 to 5 (0 is for concrete surfaces showing no sign of scaling and 5 is for a surface that is severely scaled with coarse aggregate visible over the entire surface). The subjectivity inherent in the rating scale is one problem with this test (Pigeon and Plateau 1995). As a result, it is becoming common for researchers to measure the mass of scaled material to gain a more objective measure of scaling resistance.

The Bóras method (Swedish Standard SS 13 72 44) is similar to ASTM C 672, but has been modified to more accurately represent field conditions. A freezing front passes from the top of the specimen

downward. This test allows for testing of specimens obtained from *in situ* structures as well as those prepared in the lab. A rating is assigned based on the mass of scaled material. Although similar to ASTM C 672, Pigeon and Plateau (1995) believes that it is a better test for evaluation of scaling resistance. Unfortunately, correlation between laboratory tests and field performance are not very good (Pigeon 1994; Mehta 1991).

In consideration of deicer deterioration at the joint/crack, it is obvious that the potential deterioration mechanisms are not well enough understood that specific material selection or mix design recommendations can be made. The permeability of the concrete clearly has a large impact on this distress type, and thus any changes to mix design that decrease permeability would likely positively impact the resistance to deicer deterioration. Further, it would be advantageous to alter the chemistry of the cementitious materials so that less soluble hydration products are produced. Increased production of calcium silica hydrate (CSH) at the expense of CH would likely be beneficial. Also, a reduction in aluminates may reduce the potential for the formation of calcium chloroaluminates.

Preventive Strategies for Controlling Aggregate Freeze-Thaw Deterioration

The best method of preventing aggregate related freeze-thaw problems is to use aggregates with a demonstrated service record. Aggregates associated with freeze-thaw deterioration are almost always of sedimentary origin (both calcareous and siliceous) or in a few cases sedimentary rocks that have been partially metamorphosed (Stark 1976). Although many sedimentary aggregates have been shown through both laboratory testing and field performance to provide good freeze-thaw durability, those with low bulk specific gravity, a high porosity, and a large number of small pores are often susceptible to freeze-thaw deterioration (Pigeon and Plateau 1995). Aggregates of igneous and metamorphic origin typically have good freeze-thaw performance because of their very low porosity.

Laboratory testing of aggregates for freeze-thaw durability has been generally grouped into two categories (Larson and Cady 1969). In the first group are tests that attempt to predict field performance by simulating the service environment to which the concrete will be exposed. In the second group are tests that are based on correlating an identifiable and measurable characteristic of the aggregate with known field performance.

The first group includes tests that subject aggregate or concrete specimens to various freezing conditions. These include the rapid freezing and thawing of concrete (ASTM C 666) and the critical dilation test (ASTM C 671). Other laboratory tests include the Washington hydraulic fracture test, which attempts to simulate the internal hydraulic pressures experienced during a freezing event by the use of a high pressure chamber, and the Kansas DOT Soundness Test for Aggregates.

Included in the second group are tests that attempt to measure either directly or indirectly some characteristics of the aggregate pore system. These include the Iowa Pore Index test, mercury intrusion porosimetry measurement of pore size and volume, and microscopic characterizations of the pore system. Because the freeze-thaw durability of an aggregate is not an intrinsic property of the aggregate alone, but includes the confining environment of the mortar in which it is contained and the environmental conditions to which it is exposed, it is reasonable to group test procedures by whether they are confined or unconfined.

ASTM 666, *Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing*, is the most common laboratory procedure currently used to assess the expected freeze-thaw performance of concrete aggregates. The details of this procedure have already been described. One variation used by both Kansas and Iowa when testing aggregate performance is an extended moist curing time of 90 days (Schwartz 1987). This is to help ensure that the paste phase has time to fully develop before testing. Michigan uses pressure saturation as a pre-treatment of the coarse aggregates prior to casting test specimens to ensure that the aggregates have an opportunity to become critically saturated during the test cycles. Although ASTM C 666 does provide for the calculation of a durability factor, it does not

indicate what value of the durability factor is acceptable. A number of agencies have found that increasing the number of test cycles for acceptance from 300 to 350 provides a better distinction between good and poor aggregates (Schwartz 1987). Depending on the length of curing and the number of cycles, this test can take as long as 5 months to complete.

ASTM C 671, *Standard Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing*, is also called the Powers Slow Cool Test. Concrete test cylinders (75 mm by 150 mm) are kept submerged in water at 1.7°C and every 2 weeks they are placed in water-saturated kerosene and the temperature is lowered at 2.8°C per hour from 1.7°C to -9.4°C. During the cooling cycle, specimen length changes are measured as a function of time. Specimens are then returned to their water bath until the next cooling cycle. The specimen dilation, caused by internal ice formation, is obtained from a plot of the specimen length change versus time. Specimens made with durable aggregates typically exhibit a small dilation that remains nearly constant after many cycles while non-durable aggregates result in large dilation that significantly increases with repeated cycles. A critical dilation is reached, and the specimen is considered significantly damaged when the dilation increases by a factor of two between successive cooling cycles. This procedure does a good job of distinguishing very good durability concrete from very poor durability concrete, but typically does not provide an assessment of marginal concrete that deteriorates slower (Schwartz 1987).

The Washington Hydraulic Fracture test uses a high pressure chamber in an attempt to simulate the hydraulic pressures experienced by aggregates during a freezing cycle (Janssen and Snyder 1994). An unconfined aggregate sample of known size range and number of particles is surface-treated and placed in a pressure chamber filled with water. A pressure of 7930 kPa is applied to the aggregates, forcing the water into the pore system under pressure. The pressure is rapidly released, causing the internal water in the aggregate pore system to be forced back out by the compressed air within the aggregates. Fracturing of the aggregates occurs if the pore pressure is not dissipated quickly or if the internal pressure cannot be elastically accommodated. After 10 cycles of applying and releasing pressure, the aggregate sample is removed, dried, and tumbled in a rock tumbler and graded and counted. This process is repeated 5 times for a total of 50 cycles and a percentage of fractures are used to calculate a hydraulic fracture index. Although this procedure can identify aggregates likely to fail due to hydraulic pressures, it does not simulate the confining effect of the paste phase and cannot identify aggregates that cause paste failure due to excessive water expelled from aggregates.

If aggregate freeze-thaw deterioration is a potential problem as revealed either through experience and/or the results of aggregate testing, preventive measures must be taken. Obviously, the best method of preventing aggregate freeze-thaw deterioration is to reject the use of susceptible or marginal aggregate, and use only aggregate with demonstrated good field performance and/or laboratory testing results. Unfortunately, this is not feasible in many areas where sources of non-susceptible aggregate do not exist or where laboratory test results are ambiguous.

Therefore, if a marginal aggregate source is being considered for use, it must undergo benefaction to reduce the susceptibility to this MRD. To this end, three methods have been used with some success. The first method attempts to separate susceptible aggregate from non-susceptible aggregate using the aggregate bulk specific gravity. It has been found in some cases that aggregates having lower bulk specific gravities are less resistant to freeze-thaw deterioration. This difference can be used to separate aggregate using heavy-media separation (Schwartz 1987). With the elimination of the lighter aggregate particles, the remaining aggregates are considered to be more resistant to freeze-thaw deterioration. Although this method is workable, it is not considered to be the most effective benefaction method.

Another method to improve the freeze-thaw resistance of an aggregate source is to blend it with an aggregate source that is known to be frost-resistant. Blending simply dilutes susceptible aggregate with non-susceptible aggregate, increasing the overall freeze-thaw durability of the concrete. The exact percentage of blending will vary with the aggregate source. Unfortunately, this method is not considered to be very effective, as it has a tendency to simply delay the onset of damage rather than preventing it, although it may reduce the severity of the distress once it occurs (Schwartz 1987).

The third method is to reduce the maximum aggregate size below the critical size needed to cause damage. This method has become standard practice in many highway agencies and overall has been effective in reducing the incidence of aggregate freeze-thaw deterioration. The degree that the aggregate size must be reduced varies with the aggregate type. In many instances, the maximum aggregate size has been reduced to 19.5 mm, and in some cases as small as 12.5 mm. Because of the ease of applying such criteria, it has become common practice in some agencies to specify mixtures having small maximum aggregate size even if the aggregate is not demonstrated to be freeze-thaw susceptible. This practice, although seemingly effective in reducing aggregate freeze-thaw deterioration, has led to a significant increase in the paste requirement for pavement concrete. It also may have detrimental effects on the mechanical behavior of the concrete pavement (particularly in the load transfer efficiency across cracks and non-doweled joints).

With the desire to use a larger maximum aggregate size, the best approach might be to use a combination of blending and size reduction for aggregate benefaction. Aggregate sources with demonstrated susceptibility to freeze-thaw deterioration can be crushed to smaller, non-susceptible sizes, and then blended with larger aggregate obtained from a non-susceptible source. This combination of techniques should provide a good compromise between enhanced durability at minimal additional cost, as only the larger size fractions need to be purchased and shipped.

Regardless of the benefaction method employed, the effectiveness should be assessed in the laboratory prior to construction. It is believed that the laboratory test method must confine the aggregate in a concrete matrix, such as ASTM C 666 and ASTM C 671. Both tests require expensive equipment and considerable time. But these tests can be used to establish the aggregate freeze-thaw resistance in advance of construction, then correlated with a more rapid test such as ASTM C 295.

Preventive Strategies for Controlling Alkali-Silica Reactions

In areas with known ASR problems or if testing of the aggregate indicates that ASR potential exists, preventive measures should be exercised to prevent ASR in the field. Deleterious ASR can be prevented either through the reduction of reactive silica, the lowering of the pH of the pore water, the reduction of available moisture, or through chemically altering the concrete environment. An overview of each of these methods is included below. An excellent resource available to assist in the prevention of ASR is a document entitled, *Guide Specifications for Concrete Subject to Alkali-Silica Reactions* (PCA 1995). ACI 221.1R-98 also provides a good overview of preventative measures that can be employed to mitigate ASR. Both references provide a detailed description of techniques and test methods that can be used to minimize the risk of suffering deleterious alkali-silica reactions in concrete.

Two methods have recently been put forward to address ASR problems. The proposed Canadian standard, as described by Fournier et al. (1999), considers not only the degree of aggregate reactivity, but also the environment in which it will serve, the level of risk, and the design life of the structure to determine a desired level of prevention. Five different levels of prevention are considered, ranging from "Nothing Special" to "Exceptional Preventative Action Required." Preventative measures range from accepting the aggregate in the case of a "Nothing Special" prevention level to rejecting the aggregate, limiting the total alkalis in the mixture, or using supplementary cementitious materials in the other cases. There is no testing of the mixture specified to verify the effectiveness of the mitigation.

The Proposed AASHTO Guide Specifications (AASHTO 2000) advance a slightly different approach, in which an ASR susceptible aggregate is either rejected or mitigated. Mitigation is absolute in the sense that mixture proportions must be altered until the prevention criteria are met. There is no consideration for level of prevention required (based on risk level and design life). Instead the proposed standard adopts a "zero-tolerance" approach for deleterious ASR, regardless of the pavement's location or level of service. Thus, concrete used in a local road in a desert would have to meet the same prevention criteria as that going into a multi-lane, urban interstate in a wet-freeze climate. The mitigation techniques considered include the use of low alkali or blended cements, the use of supplementary cementitious materials, or lithium admixtures.

If possible, field performance history is the best method for determining the potential for ASR in concrete mixes. In order to compare field performance of an older mix with potential field performance of a suggested mix, several factors must be considered:

- Are the cement content, w/c , and alkali content of the cement the same or higher than that of the proposed cement?
- Is the field concrete at least 15 years old?
- Are the exposure conditions the same?
- Were pozzolans or slags used in the field?
- Are the aggregates petrographically similar?

If the answer to all of the above questions is "yes," then the field concrete is representative of the proposed concrete mixture (Farny and Kosmatka 1997; CCAA 1996). Commonly, differences between mixtures placed in the field many years ago and new mixtures make it impossible to base mix design decisions on performance of existing pavements. For this reason, other methods must be used.

One obvious method to prevent ASR is to reject the susceptible aggregate source in favor of aggregate not containing reactive silica. This method is particularly warranted if the aggregate is found to be highly reactive, in which case other preventive treatments may not be effective. But in many cases, this may not be an option as less reactive aggregate sources may not be available. It is also possible to blend aggregate containing reactive silica with non-reactive aggregate, effectively reducing the total amount of reactive silica in the concrete. Limestone "sweetening," for example, entails replacement of up to 30 percent of reactive fine aggregate with crushed limestone, effectively reducing deterioration in some sand-gravel mixes (Farny and Kosmatka 1997). But in some cases, reducing the amount of reactive silica can actually make ASR more severe. Although there is less reactive silica present, the alkali content remains constant and as a result, the alkali content to reactive silica ratio increases, resulting in a more vigorous reaction. Therefore, when using limestone sweetening, the mix should be tested according to ASTM C227 or ASTM C1293 (Farny and Kosmatka 1997).

Collins and Bareham (1987) found that expansion due to ASR was reduced when porous natural or lightweight aggregate replaced limestone in a concrete mix. There was no evidence of the gel filling in the pores of the aggregate. The reduced expansion is believed to be caused by the dilution of the alkalinity in the pore solution, which resulted from water being absorbed into the aggregate. This hypothesis was mildly supported by the ASTM (1990), which pointed out that changes in the reactivity of aggregate can be attributed at times to variations in the aggregate porosity.

Assuming that a marginally reactive aggregate is being considered for use, deleterious ASR can be prevented through the application of a number of preventive strategies. One strategy is to reduce the available alkalis in the pore solution. Typically, the use of low alkali cement having a total alkali content of less than 0.6 percent is considered effective in controlling ASR when used with low to moderately reactive aggregates. If highly reactive aggregates are being used, the low alkali cements have been less effective (Farny and Kosmatka 1997). More importantly, the total alkalis from all sources in the mix should be considered, including not only the cement, but also other concrete constituents including fly ash, aggregate, admixtures, and mixing water. In situations where potentially reactive aggregate must be used, the proposed Canadian standard (Fournier et al. 1999) stipulates that the total alkalis be limited to a maximum of 3.0 kg/m³ Na₂O_e for mild prevention to 2.2 kg/m³ and 1.7 kg/m³ Na₂O_e for moderate to strong preventative action, respectively.

In hot climatic regions where highly reactive aggregates are present, such as in the southwestern United States, reducing the total alkali content in the concrete may not be adequate. In such cases, one or more additional actions will be required. These include (Gress 1997):

- Use of a Type IP cement.
- Addition of lithium.

- Addition of high silica, low calcium fly ash (ASTM Class F).
- Addition of granulated, blast-furnace slag.
- Addition of silica fume.

Blended hydraulic cements must conform to ASTM C 595 or ASTM C 1157 to control ASR. If raw or calcined natural pozzolans are being used, ASTM C 618 should be met. It has been found that higher amounts of slag are needed to control ASR than fly ash. Silica fume must conform to ASTM C1240 and is only needed in a small amount (Farny and Kosmatka 1997; CCAA 1996).

High silica fly ash has been found to be effective in reducing expansion due to ASR. It is believed that the cement alkalis are diluted by fly ash having a lower alkali content, some alkalis are bound to the hydration products, and there is a reduction in concrete permeability due to pozzolan behavior of the fly ash (Detwiler 1997). Keck and Riggs (1997) and Sturup et al. (1983) also believe that the silica component of the fly ash consumes the available alkali in the cement, reducing the level of expansion.

Farny and Kosmatka (1997) believe that simply using a lower w/cm will create a lower relative humidity within the concrete and, as a result, less gel expansion will occur. A low w/cm also reduces the concrete permeability, limiting the movement of alkalis and moisture within the paste.

As previously mentioned, lithium has recently been employed as a concrete additive to reduce ASR expansion (Gajda 1996; ACPA 1993). Lithium ions in the pore solution of the concrete inhibit the formation of soluble ASR gel (Wang et al. 1994). The ability of lithium to inhibit ASR depends on the compound used, addition rate, aggregate reactivity and cement alkalinity (TRB 1999). The proposed AASHTO Guide Specification (AASHTO 2000) provides considerable information on the use of various lithium compounds and recommended addition rates. It has been found that ASTM C1260 is not effective in determining whether lithium compounds are effective (Farny and Kosmatka 1997).

Regardless of the approach being used to mitigate ASR, it is desirable to test the reactivity of the mixture with all constituent materials combined according to the job mix formula. In the proposed AASHTO Guide Specifications, it is suggested that ASTM C 441, ASTM C 1260, and ASTM C 1293 be used to test mixtures containing all the constituent materials proportioned according to the job mix formula to verify mitigation. For each test, expansion criteria are set along with limitations. For example, the expansion limit using ASTM C 1260 is 0.08 percent for metamorphic aggregates and 0.10 for all other aggregates, yet the test can not be used to evaluate the effectiveness of treatment with a lithium compound.

Unfortunately, accelerated tests as documented in the ASTM standards, depend either on using a highly reactive aggregate or a highly alkali environment to shorten the time frame for analysis. For example, ASTM C 441 is commonly used to assess the effectiveness of mineral admixtures or slag in reducing expansion due to ASR, but it does not use the aggregate under consideration (pyrex glass aggregate is used). Farny and Kosmatka (1997) recommend using ASTM C 1260 to determine if fly ash or ground granulated blast furnace slag (GGBFS) are effective in reducing the ASR, but they note that this test cannot be used to judge the effectiveness of reducing total alkalinity or adding lithium compounds to the mixture. In addition, it is noted that the mechanism responsible for the reduction in expansion that occurs in either ASTM C 441 or C 1260 when used to test the effectiveness of GGBFS and/or a pozzolan might not be the same as would occur in the field. This is because the test duration of 14 days is too short for the slag and/or pozzolan to fully react (ACI 1998). In the Canadian standards, it is recommended that a 2-year test be run in accordance with ASTM C 1293 to fully assess the ability of slag and/or pozzolans to mitigate ASR. Research continues to establish correlation between laboratory tests and field performance.

Until correlation between laboratory tests and field performance is established, the best strategy might be to use local experience to develop mitigation strategies that are known to work. This is the approach taken in the proposed Canadian standards, where testing of the combined constituent materials is not done. The approach instead depends on the testing of the aggregates to establish the degree of

reactivity, preferably using ASTM C 1293, although ASTM C 1260 results can also be used. Once the degree of reactivity is established, the level of risk can be assigned to determine the level of prevention needed. Preventive measures are then applied accordingly, with no additional testing.

In closing, research into ways to mitigate ASR is continuing. Two new approaches have been recently proposed for mitigating ASR if reactive aggregates will be used. The mitigation techniques focus on limiting total alkalis in the mix, the use of supplementary cementitious materials, or the addition of lithium compounds. Other methods are also available, including aggregate benefaction.

Preventive Strategies for Controlling Alkali–Carbonate Reactions

The surest way to prevent ACR is to screen aggregates. A number of test methods are available for screening, including field performance, petrographic examination, the rock cylinder test, and the concrete prism test (ACI 1998). Petrographic examination of aggregate using ASTM C 295 is a very useful tool in identifying the composition of the aggregates proposed for the mix design. ACR-susceptible aggregate has very characteristic features, namely dolomitic limestones having a fine grained matrix of calcite and clay surrounding rhombic crystals of dolomite. Descriptions of aggregate that may cause ACR are presented in ASTM C 294. Thus, using petrographic examination, a designer can determine if aggregates possess the characteristics necessary for ACR (Farny and Kosmatka 1997).

Physical expansion tests, such as the rock cylinder method (ASTM C 586), allow the potential for expansion of the aggregate to be nominally measured. The test measures the expansion of small cylinders of rock during exposure to sodium hydroxide solution (Ozol 1994). Mingshu et al. (1994) and Farny and Kosmatka (1997) support the use of the rock cylinder method to aid in the assessment of ACR potential. It is not recommended that the rock cylinder method be used as the sole test to judge an aggregate's potential reactivity, but rather as a supplement to other testing methods (Ozol 1994).

The concrete prism test, ASTM C 1105, was developed specifically for use with carbonate aggregates to evaluate ACR susceptibility. Similar to the rock cylinder method, the concrete prism test measures the expansion potential of carbonate aggregates in a high alkali solution (Farny and Kosmatka 1997; Ozol 1994). Bérubé and Duchesne (1992) and Ozol (1994) believe that the concrete prism method is the best method for testing aggregates for ACR potential. Bérubé and Duchesne think that ACR-susceptible aggregates do not show appreciable expansion when other methods are used, specifically discouraging the use of the Pyrex mortar bar method (ASTM C 441) in determining ACR potential.

Because ACR expansion can be slow, Gillott (1995) suggests the use of field evidence, when possible, to determine the ACR potential of an aggregate. He warns that certain criteria must be met prior to using field evidence as a measure of aggregate reactivity. The same aggregate must be used in the field sample as is used in the proposed mix design. Similarly, the same type of cement (or one with similar alkali content) and mix design (including cement and admixture content and proportions) should be used in the field test. The conditions of exposure of the field concrete and the proposed structure should be similar. Finally, the age of the field concrete should be considered such that the alkali–carbonate reaction must have had time to develop. Ozol (1994) adds that if the structure shows no distress in the field and the alkali content is high enough to cause ACR, the aggregate cannot be characterized as inert until the aggregate size, proportion of reactive aggregates, and moisture availability are also considered.

If a reactive rock source is identified, selective quarrying can be used to isolate reactive rocks so that they are either not used at all or they are used in limited amounts. If potentially reactive aggregate must be used, it is possible to blend them with non-reactive aggregate to limit the potential for deleterious ACR. Farny and Kosmatka (1997) recommend that the reactive aggregate should be limited to 20 percent by volume if either the coarse aggregate or the fine aggregate is reactive. If both the coarse and the fine aggregate are reactive, then only 15 percent by volume of the aggregate used should be reactive. The maximum size of the aggregate also affects its reactivity. The larger the maximum size of the aggregate, the more potential it has to be reactive (ACI 1998).

The use of low alkali cements is far less effective in controlling ACR than ASR. Alkalies are regenerated through the alkali carbonate reactions, and even cement alkali content as low as 0.40 percent can still lead to initiation of this distress (Farny and Kosmatka 1997). This is in contrast to literature cited in ACI (1992), which suggests that the use of low alkali cement will prevent harmful expansion in most cases. The most important cement parameter to consider when potential ACR susceptibility exists is to minimize the total alkali content per unit volume of concrete, and not focus exclusively on the alkali content of the cement (Ozol 1994).

Pozzolans are also not effective in controlling ACR. However, preliminary evidence suggests that lithium-based admixtures appear to control ACR, although little research has been performed to support this finding (Farny and Kosmatka 1997). Thus, the only demonstrated method currently available to prevent ACR is to eliminate or significantly reduce the amount of reactive aggregate.

Preventive Strategies for Controlling External Sulfate Attack

The need for high quality, impermeable concrete is a prerequisite for concrete resistance to external sulfate resistance. Concrete with a low w/cm and high cement factor is consistently recommended, as it will have lower permeability and thus limit the amount of sulfate ions that can diffuse into the concrete to attack it. This requires good workmanship, curing, and a relatively rich mix with a low w/cm . It is thought that air entrainment is beneficial only in that it makes the concrete more workable, so the w/cm ratio can be reduced. Leek et al. (1995) give recommendations for general chemical attack resistance. They recommend that minimizing voids and cracks, ensuring a good bond between aggregate and cementitious paste, minimizing porosity of the paste, and minimizing the paste fraction of the concrete can all improve resistance to chemical attack through decreased permeability. This approach is also advocated by ACI (1992), which reports that good sulfate resistance can only be ensured by reducing the permeability of the concrete through a low w/cm and good curing practices.

The contribution of the cement type and composition to resistance to external sulfate attack has already been discussed at some length. The recommendations made in table 1-10 should be followed when selecting a cement and w/cm for concrete to be used in a sulfate rich environment. Tuthill (1993b) cites the U.S. Bureau of Reclamation in recommending that the soil and soil moisture be tested for sulfates, unless there is a good service record of nearby concrete in the ground. If the groundwater contains up to 1500 ppm of SO_3 , Type V cement is recommended. If the sulfate concentration is higher, either an extra sack of Type V portland cement per cubic yard or else a 25 percent replacement with fly ash is recommended. If more than 10,000 PPM of SO_3 exist, it is recommended to use both additional cement and fly ash replacement.

Table 1-10. Recommendations for normal weight concrete subject to sulfate attack (ACI 1992a).

Exposure	Water Soluble Sulfate ¹ (SO_4) in soil, percent	Sulfate ¹ (SO_4) in water, PPM	Cement	Water-cement ratio, maximum
Mild	0.00 to 0.10	0 to 150	—	—
Moderate ²	0.10 to 0.20	150 to 1500	Type II, IP(MS), IS(MS) ³	0.50
Severe	0.20 to 2.00	1500 to 10,000	Type V ⁴	0.45
Very Severe	Over 2.00	Over 10,000	Type V and pozzolan or slag ⁵	0.45

¹Sulfate expressed as SO_4 is related to sulfate expressed as SO_3 as in reports of chemical analysis of cement as $SO_3 \times 1.2 = SO_4$.

²When chlorides or other depassivating agents are present in addition to sulfate, a lower water-cement ratio may be necessary to reduce corrosion potential of embedded items.

³Or a blend of Type I cement and a ground granulated blast furnace slag or a pozzolan that has been determined by tests to give equivalent sulfate resistance.

⁴Or a blend of Type II cement and a ground granulated blast furnace slag or a pozzolan that has been determined by tests to give equivalent sulfate resistance.

⁵Use a pozzolan or slag that has been determined by tests to improve sulfate resistance when used in concrete containing Type V cement.

Although the C_3A content of cement is the most common cement parameter related to external sulfate attack, DePuy (1994) cites some exceptions where low C_3A cements have done poorly, and some moderately high C_3A cements have had good sulfate resistance. As described, some researchers also believe that C_4AF must also be considered. For this reason, the Type V cement has a maximum calculated C_3A content of 5 percent and a combined $C_4AF + 2C_3A$ content that does not exceed 25 percent. Van Aardt and Visser (1985) determined that the alkali content of the cement could also influence the sulfate resistance of concrete. They found that the following ratios give preliminary indications of resistance to sulfate attack when both were satisfied:

$$\frac{C_3A\%}{SO_3\% + Na_2O\ eq\%} < 3$$
$$1 < \frac{SO_3\%}{Na_2O\ eq\%} < 3.5$$

(1-8)

For Type V cement, ACI allows the sulfate expansion test, ASTM C 452, to be used in lieu of the chemical requirements. If the proposed cementitious binder is a portland cement or a portland cement blended or modified with slag or fly ash, its sulfate resistance can be tested using ASTM C 1012 (DePuy 1994). Mehta (1991) notes that this test cannot simulate the service behavior of concrete and is of limited value in predicting field performance. Stark (1989) reports that in the low and high w/cm mixes, cement type made little difference, but in the intermediate w/cm mixes (0.45 to 0.55), cement type made significant differences.

Concrete with a low w/cm and high cement factor is consistently recommended, as it will have lower permeability that will limit the amount of sulfate ions that can diffuse into the concrete (ACI 1992a; Emmons 1994; Hognestad 1993; Mielenz 1962; Stark 1984). This is confirmed by DePuy (1994), who cites numerous reports stating that high-quality, impermeable concrete is more resistant to sulfate attack. The ACPA (1993) and Mehta (1991; 1997) report that having a high quality concrete with adequate cement, proper thickness, proper curing, and low w/cm is the best protection against sulfate attack, with the composition of the cement of secondary importance. This requires good workmanship, curing, and a relatively high cement content mix with a low w/cm . It is thought that air entrainment is beneficial only in that it makes the concrete more workable, so the w/cm ratio can be reduced. This was confirmed by Tuthill (1993a), who found that air entrainment should not be used for primary resistance to sulfate attack.

Concrete permeability is a key parameter when considering external sulfate attack, with reduced permeability resulting in improved resistance. In one study, Mangat and Khatib (1995) found that air curing at 20°C and 55 percent RH for 28 days provided the best resistance to sulfate attack for both fly ash and ordinary concrete. The dry curing was as effective as replacement of cement with fly ash, silica fume, or slag. They theorized that this might be due to a carbonated layer formed at the concrete surface

under the dry curing conditions. However, greater depths of carbonation did not result in greater sulfate resistance in all cases. Mehta (1991) notes that limited carbonation of paste will improve impermeability due to the formation of insoluble calcium carbonate. Yet excessive carbonation in the presence of aggressive CO_2 in soft water will increase permeability. Soluble calcium bicarbonate will be formed and leach away, widening the existing microcracks and pores.

The amount of CH in the cement paste has an effect on the sulfate resistance as it is involved in gypsum corrosion. For this reason, pozzolans are effective in improving resistance to sulfate attack in severe sulfate environments where Type V cement alone may not give adequate protection. In these conditions, it is recommended that a Type VP or VS cement be considered. Supersulfated slag cements, if available, are also a potential option.

A tremendous amount of literature is available on the impact of fly ash on sulfate resistance. Klieger and Gebler (1987) found that concrete made with 25 percent replacement of Class F fly ash showed less expansion due to sulfate attack than that made with Class C fly ash. They called those fly ashes with less than 10 percent CaO Class F, and those with greater than 10 percent CaO Class C, although the fly ashes used did not comply with all of the chemical specifications for ASTM C 618-83. Torri et al. (1995) found no deterioration after 2 years with a 50 percent replacement using Class F fly ash. Two fly ashes were used, one ground finer than the other. No difference was found between the two. The fly ashes were mixed in a 400 kg/m^3 binder content concrete.

Ellis (1993) cited several studies that showed Class C fly ash usually had a negative effect on sulfate resistance of concrete compared to Type II cement, unless used at levels of 40 to 50 percent of the total cementitious content. Class C fly ash contains relatively high levels of amorphous calcium aluminate, which decreases performance in a sulfate environment. Stark (1989) reported little effect on sulfate resistance from the Al_2O_3 content of fly ash. Class C fly ashes often contain additional crystalline phases such as tricalcium aluminate, lime, and anhydrite, which are sources of alumina, calcium, and sulfate for the sulfate attack (Tikalsky et al. 1992). Additionally, Class C fly ashes have a higher CaO content, which reduces their ability to convert CH to CSH during hydration. The high calcium fly ashes are in themselves cementitious, leaving behind much calcium hydroxide to later react with sulfates. Thus, Class C fly ash generally does not perform well in high sulfate exposure conditions. McCarthy et al. (1989) gave preliminary results indicating that sub-bituminous fly ash with greater than 25 percent CaO was doing very poorly in sulfate tests. Tikalsky et al. (1992) found that Class C fly ashes were detrimental to the sulfate resistance of the concrete tested.

Freeman and Carasquillo (1996) conducted a study involving Class C fly ash and anhydrous sodium sulfate as a chemical admixture during mixing. They found that adding the sodium sulfate in the proper amount reduced the permeability and improved the sulfate resistance of the concrete, making it more resistant than Type II cement.

In summary, Class F fly ash is generally found to be beneficial in improving sulfate resistance, whereas Class C fly ash may actually be detrimental. For these reasons, only high quality, Class F fly ash should be considered for use in improving sulfate resistance of concrete. It is thought that fly ash meeting ASTM C 618 and having less than 10 percent bulk CaO can be used to improve sulfate resistance. Fly ash containing 10 to 25 percent CaO should be tested using the actual materials to be used in the concrete.

The addition of GGBFS also has beneficial effects toward sulfate durability. According to ACI (1987), the substitution of slag reduces the C_3A content by reducing the amount of portland cement in the concrete. It also reduces soluble calcium hydroxide in the formation of CSH, compromising the environment required for the formation of ettringite. CSH also forms in pore spaces normally occupied by alkalis and calcium hydroxide, reducing the permeability of the paste. Slag has latent hydraulic and pozzolanic properties, while fly ash has only pozzolanic properties. The clinker and slag reaction creates semi-permeable membranes in the pores between slag or pozzolan and clinker particles, further decreasing permeability (Bakker 1983). Studies have found that the initial permeability of GGBFS blended cements is higher than

that of ordinary portland cement, but that with prolonged curing, the GGBFS ultimately has a lower permeability due to long-term hydration (Gowripalan et al. 1993).

DePuy (1994) and ACI (1987) cite studies that also show blended slag concretes have improved sulfate resistance when GGBFS is substituted for at least 50 percent of the cement and when the Al_2O_3 content is limited. If the alumina content of the GGBFS is less than 11 percent, increased sulfate resistance was found regardless of the C_3A content of the cement (ACI 1987). Stark (1989), on the other hand, found little significance in the Al_2O_3 content of the GGBFS.

When using pozzolans or supplementary cementitious materials, a comprehensive materials characterization analysis should be done using X-ray diffraction, bulk composition analysis, and analytical glass composition. The glass composition is used with the ternary diagram to determine the reactivity of the glass phase with regard to long-term resistance to sulfate attack. DePuy (1994) gives the following general guidelines for selecting pozzolans that increase sulfate resistance:

- Class N pozzolans are variable, but generally good. Best results are obtained if replacement is limited to 15 percent.
- Most Class F fly ashes improve sulfate resistance of Type II and V cements. They generally work better than Class C fly ash, but some with high alumina content may not be as effective.
- Low calcium Class C fly ashes are often good, but high calcium Class C fly ashes are variable and may reduce sulfate resistance. Some Class C fly ashes must be used at greater than 75 percent replacement to be sulfate resistant.
- Silica fume is generally good for improving sulfate resistance, possibly more effective than fly ash or ground slag.

The R-factor, first contrived by Dunstan (1980; 1987) and cited in many papers, may be useful in assessing the ability of a fly ash to improve sulfate resistance. It is as follows:

$$R = \frac{(CaO \text{ percent} - 5)}{(Fe_2O_3 \text{ percent})} \quad (1-9)$$

Fly ashes with R-factors less than 1.5 improved the sulfate resistance of Type II cement, while R-factors over 3.0 were detrimental. Fly ashes between 1.5 and 3.0 had no significant influence on sulfate resistance. Table 1-11 provides an interpretation of the R-factor versus improvement in sulfate resistance (Mangat and Khatib 1995).

Table 1-11. Improvement in sulfate resistance versus the R-factor (Mangat and Khatib 1995).

R-factor	Sulfate Resistance
< 0.75	Greatly improved
0.75 to 1.5	Moderately improved
1.5 to 3.0	No significant change
>3.0	Reduced

Hartmann and Mangotich (1987) provides several problems with the R-value. They state that it cannot be used to predict relative performance of concrete with portland cement only, or the relative performance of different types or brands of portland cement. It also fails to predict the improvement in sulfate resistance observed with some high lime fly ashes. Also, some high R ashes improved resistance beyond some low R ashes, contrary to what had been predicted. McCarthy et al. (1989) also criticize the R-factor, stating that Dunstan recognized that certain chemical compounds, such as C_3A , C_4A_3S , and gehlenite-rich glass were likely part of the problem of sulfate attack, but the R-factor does not measure these. Nor does it measure the alumina content of the fly ash.

In trying to address some of the limitations in the R-factor, Hartmann and Mangotich (1987) created the Oxide Durability Factor (ODF) from results of expansion data on mortar bars. It relates the chemical compositions of the cementitious ingredients. The formula is given as:

$$O D F = \frac{(Total CaO)(Free CaO)}{SiO_2 + Al_2O_3 + Fe_2O_3} \quad (1-10)$$

The ODF was found to vary directly with the sulfate durability of the concrete. The ODF was checked against the work of other experimenters and found to be a good predictor of sulfate resistance.

Tikalsky et al. (1992) found that fly ash with a CaO/SiO_2 ratio less than 0.25 and containing no detectable C_3A at a 35 percent volume replacement provided resistance greater than Type II cement alone. Fly ash containing detectable C_3A and a CaO/SiO_2 ratio greater than 0.6 decreased sulfate resistance. Unfortunately, these limits would eliminate over 90 percent of fly ash available for use in concrete.

The replacement of portland cement with GGBFS also has beneficial effects toward sulfate resistance through the reduction of the C_3A content incurred by reducing the amount of portland cement in the concrete. It also reduces soluble calcium hydroxide by increasing the formation of CSH, compromising the environment required for the formation of ettringite. CSH also forms in pore spaces normally occupied by alkalis and calcium hydroxide, reducing the permeability of the paste.

The sulfate resistance of concrete is decreased through the addition of calcium chloride. It therefore should not be added to concrete subjected to severe or very severe sulfate exposure conditions.

Due to variability in the effectiveness of various techniques to improve sulfate resistance, it is important that specific combinations of the cement and pozzolan be tested to verify sulfate resistance. When using pozzolans or GGBFS with Type V cement, the combination to be used should be tested with an accelerated testing procedure. This is because the low alkali content of Type V cements may not activate the pozzolanic ingredients in the blended cements (ACI 1992a). ASTM C 1012 can be used to assess the sulfate resistance of blended cements or cement-pozzolan mixtures.

Unfortunately, assessing the sulfate resistance of concrete is difficult. There is currently no standard ASTM test for assessing the sulfate resistance of specified concrete made using the selected constituent materials and job mix formula. ASTM C 452 evaluates only the sulfate resistance of portland cement and not that of the concrete. ASTM C 1012 is the most commonly recommended test to assess the sulfate resistance of portland cement, blends of portland cement with GGBFS and fly ash, or blended hydraulic cements. Six-month expansion limits of 0.10 and 0.05 percent roughly translate to moderate sulfate resistance and high sulfate resistance, respectively. But it too only tests the resistance of the cementitious materials and not the concrete. Modifications to the standard test methods could be made so that the job specific concrete is tested. This would be particularly useful as mix parameters are considered to be at least as influential as cement chemistry in the sulfate resistance of concrete.

Preventive Strategies for Controlling Internal Sulfate Attack

It is uncertain to what degree internal sulfate attack affects pavements and, as a result, it is difficult to make specific recommendations to prevent it. However, in light of some well-known premature pavement failures that have occurred in recent years (Gress 1997; Moss et al. 1997), it seems prudent to consider internal sulfate attack as a potential problem in pavements. Prevention must focus on the evaluation of the concentration and chemical form of sulfur within constituent materials (cement, aggregate, and admixtures), potentially establishing limits to control this problem. Concrete permeability may also have an influence as a reflection of the ease of transport for various species within the concrete matrix.

For internal sulfate attack, the composition of the cement is potentially an important factor. Sulfate-related deterioration in concrete is generally related to the C_3A content of the cement, with more deterioration typically occurring with higher C_3A content. Cement with a C_3A content of less than 5 percent is typically considered to be sulfate resistant. Type II and III cements containing less than 8 percent C_3A have moderate sulfate resistance. Type V cements with less than 5 percent C_3A and less than 25 percent $[C_4AF + 2(C_3A)]$ or $[C_4AF + C_2F]$ and Type III cements with less than 5 percent C_3A have high sulfate resistance (ACI 1992a).

Gress (1997) believes that limiting the sulfate content of cement is an extremely important step in preventing potential DEF. To control DEF, he recommends that a maximum sulfate content for cement and cementitious materials be limited to no more than 3 percent, although he admits that this alone may be insufficient to eliminate deleterious DEF. It is desirable to control the C_3A content as well, but a set limit is not practical due to an incomplete understanding of the DEF mechanism at this time. Suggested criteria of limiting the C_3A/SO_3 ratio to less than 2.0 to 2.5 are problematic, as it would eliminate a significant number of currently available cements. As a result, Gress concedes that it is impossible at this time to recommend a cement specification to prevent DEF.

ASTM C 1038, *Expansion of Portland Cement Mortar Bars Stored in Water*, has been used to investigate the expansion of concrete resulting from calcium sulfate (gypsum) in the cement. It is applicable to the study of a specific portland cement and thus cannot be used to consider the influence of aggregate or admixtures. An expansion limit of 0.20 percent at 14 days of water immersion is used in Canadian Standards document CAN 3-A5-M83. Expansion is directly related to the amount of calcium sulfate present in the cement and thus the impact of other sources of sulfate, such as slowly soluble chemical forms of sulfur, would not be evaluated. It is thus not thought to be a reliable indicator of DEF potential. Gress (1997) suggests that a new test method needs to be developed where heat cured mortar bars are made with the proposed constituent materials and length changes are measured over time.

The Duggan Test has been proposed as a test method that provides a “rapid measure of the potential for chemical expansion in concrete” [American Railway Engineering Association (AREA) 1996]. Expansion can be due to AAR, internal sulfate attack, or other potentially deleterious reactions. In this test, 25-mm-diameter concrete cores that are 51 mm in length are subjected to prescribed wetting and dry heat cycles for 10 days. Expansion is routinely measured following the final dry heat conditioning as the specimens soak in distilled water for a period of 3 weeks. Expansion should not exceed 0.15 percent at day 20 according to AREA specifications for concrete railroad ties. Others have suggested that if the expansion

exceeds 0.05 percent after the 20 days of water immersion, additional information be gathered to determine if a deleterious chemical reaction is implicated.

The effect of adding fly ash or GGBFS to concrete on the occurrence of DEF is unknown. On the one hand, as discussed previously, pozzolans can increase concrete resistance to sulfate attack. But fly ash and GGBFS may also provide forms of sulfur that may lead to the adverse reaction. Gress (1997) examined the possible role that fly ash may have had in premature pavement deterioration, calling for a suspension of the use of combinations of portland cement and Class C fly ash that have demonstrated early distress unless they can be proven acceptable by additional testing. In addition to identifying Type C fly ash as a possible source of slowly soluble sulfur, Gress also focuses attention on the solubility of the aluminates phases in the fly ash, stating that slowly soluble Al_2O_3 may become encapsulated within the CSH gel before going into solution. If this occurred, it is speculated that it may react at a later date. He concludes that a study is urgently needed to examine the roles of SO_3 and Al_2O_3 found in some fly ash in the development of DEF/EXS in concrete.

Another potential source of slowly soluble sulfate is the aggregate. Sulfates having very low solubility in water, such as barium and lead sulfate, are relatively harmless. But some aggregate sources may contain moderately soluble forms of sulfate that may be problematic. A petrographic analysis of the aggregate, in accordance with ASTM C 295, should be used to identify potentially problematic sources of aggregate sulfates.

Aside from the properties of the constituent materials, the overall quality of the concrete may also impact its susceptibility to internal sulfate attack. Thus, recommendations made regarding the selection of quality materials, the design of a relatively impermeable mix, and the use of proper construction methods should help reduce the potential for internal sulfate attack. Specifically, using mixes with higher aggregate content and reduced heat of hydration while following recommended hot weather concrete placement practices should have a positive impact on the prevention of internal sulfate attack.

In summary, it is currently unclear as to the best approach to prevent internal sulfate attack in concrete pavements, primarily because the distress is not well understood in pavement concrete. It is thought that factors that contribute to external sulfate resistance will also contribute to the prevention of internal sulfate attack, most notably the use of low alumina cements, low calcium pozzolans, and low permeability concrete. Specific to internal sulfate attack is the desire to restrict internal sources of sulfur from cement, aggregate, and admixtures, particularly slowly soluble forms of sulfur. High ambient temperatures during construction also seem to be implicated, especially if a high early strength, high heat of hydration concrete mixture is being used.

Preventive Strategies for Controlling Corrosion of Embedded Steel

Chapter 3 in ACI 222R (1989) provides an excellent description of how to protect new concrete construction against corrosion. The following three general categories of protection are considered:

- The use of design and construction practices that maximize the protection afforded by the concrete.
- The use of treatments that penetrate or are applied to the concrete surface that prevent chloride ion penetration.
- The use of techniques that directly protect the reinforcement from corrosion.

For pavement construction, the first and last categories are of most practical merit for prevention of corrosion of embedded steel, as the use of protective coatings on load transfer devices and adequate cover over reinforcing steel should be all that is required if properly constructed, high quality concrete is used.

The most effective pavement design factor for resisting corrosion is to provide adequate concrete cover over the embedded steel. In high quality concrete, the carbonation zone is unlikely to exceed 25 mm and thus a concrete cover of 25 to 40 mm is considered adequate for many structures (Mindess and Young 1981). But tests have shown that structures subjected to severe chloride exposure should have a concrete cover of at least 50 mm (Mindess and Young 1981; ACI 1992a; Mehta 1993). Thus, reinforced concrete pavements constructed in areas exposed to chloride-based deicers should use a minimum 50-mm concrete cover. It is noted in ACI 201.2R (1992b) that to achieve the desired 50-mm cover in paving for 90 percent to 95 percent of the reinforcing steel, the specified cover should be 65 mm.

It is important that a high chloride ion concentration not be introduced into the concrete through the constituent materials. One potential source of chloride ions in the constituent materials is the aggregate. In one study cited in ACI 201.2R (1992b), quarried stone, gravel, and natural sands were found to contain chlorides of sufficient quantity that the chloride ion concentration in the concrete exceeded permissible levels. It is noted that chlorides in aggregate are not always available to the corrosion process. Thus, if concrete made with marginal aggregate has had good field performance, higher chloride levels may be tolerated. Calcium chloride accelerator is another internal source of chloride ions and its use should be discouraged if embedded steel is used. The chloride ion concentration of mixing water should also be limited. Calcium nitrite has been found to be an effective corrosion inhibitor as well as an accelerator (ACI 1992a). The nitrite ion chemically interacts with ferric oxides, making them insoluble. This reinforces the passive film, preventing the ingress of chloride ions up to a certain threshold level (PCA 1992). The cost of a corrosion inhibitor can be rather high, adding 8 percent to the cost of concrete (Mehta 1997).

The determination of the chloride ion concentration may be desired since it is an important factor in the corrosion of embedded steel. Chloride ion concentration can be determined using either of two wet chemical analyses. One is used to determine water-soluble chloride whereas the second determines the acid-soluble chloride content. In each case, a concrete specimen is ground to powder, typically using a rotary hammer to avoid the use of cooling fluids. The most common test is the acid solubility test (AASHTO T 260), in which the pulverized concrete is dissolved in nitric acid. The water-soluble method (ASTM C 1218) has seen limited use. For the same concrete, the average chloride content measured by the water soluble method is typically 75 percent to 80 percent of its acid-soluble chloride content (ACI 1989). Chloride content is most often expressed as the chloride ion (Cl^-) present as a percentage of the mass of cement. It is very important that the chloride content being reported is defined according to the method of testing.

The permeability of the concrete to chloride ion penetration can be measured in the laboratory. AASHTO T 259, *Resistance of Concrete to Chloride Ion Penetration* was a commonly used test method in which concrete prisms are subjected to ponding of a 3 percent sodium chloride solution for 90 days. The chloride ion concentration is then measured at various depths to assess the degree of ingress. Due to the long testing period, a rapid test was devised based on the electrical conductivity of the concrete. This test is designated ASTM C 1202 *Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*. As discussed previously, it has excellent correlation to the results obtained by AASHTO T 259 for a wide variety of concrete types and qualities (except for concrete made with silica fume) (Perenchio 1994). The rapidity, ease of use, and reliability make this test very attractive when investigating the chloride ion permeability characteristics of concrete. Because of its usefulness, it is recommended that this test be routinely conducted during mix design as an overall assessment of concrete permeability.

Studies cited in ACI 201.2R (1992b) have shown that for similar cover, the level of corrosion successively decreases with decreasing w/cm . For concrete exposed to very severe chloride environments, it is recommended that a w/cm of less than 0.40 be used if possible. In lieu of this, increasing the concrete cover to 100 mm is recommended for cast in place construction in a marine environment having a w/cm of 0.45. Even the most impermeable concrete can become permeable if cracking occurs. These cracks provide easy access, concentrating chlorides, water, and oxygen to form a cell that can initiate corrosion (ACI 1992a).

Torri and Kawamura (1992) studied the effect of pozzolans and supplementary cementitious materials (fly ash, blast-furnace slag, and silica fume) and environmental conditioning on the concrete pore structure and chloride ion permeability. It was concluded that although the pore structure at the surface of concrete containing mineral admixtures was adversely affected by long-term drying, in all cases, mixtures prepared with admixtures had lower chloride ion permeability than comparable mixtures made of ordinary portland cement.

One interesting study concluded that aggregate can be as much as 1,000 times more permeable than a high quality portland cement paste and thus an overabundance of aggregate may adversely affect the ability of the concrete to protect steel (Perenchio 1994). It is therefore suggested that the maximum aggregate size and aggregate permeability are important considerations when designing against corrosion. Perenchio states that the use of large coarse aggregate particles may in effect “short-circuit” the concrete cover by providing a more direct route for harmful agents to permeate into the concrete. This should be studied in detail before adopting new gradation requirements.

Even the most impermeable concrete can become permeable if cracking occurs. Thus, sound design practices should be used to minimize cracking and to keep cracks that form in reinforced pavements tight, minimizing the infiltration of deleterious agents. Construction practices should be used that prevent plastic shrinkage cracking of the pavement surface through the application of an approved curing compound at the appropriate time. This is most important during dry, windy days when the evaporation of bleed water can be quite rapid. Poor concrete consolidation must also be avoided through the design of a workable mix and care in selecting the appropriate depth, spacing, and operating frequency for internal vibrators.

In rare cases, it may be determined that an existing concrete pavement is at risk of suffering widespread corrosion before active corrosion has actually taken place. For example, a pachometer survey conducted soon after construction may reveal that the steel was improperly placed and adequate cover does not exist. In such a case, an agency may want to take preemptive measures to prevent the future occurrence of corrosion. Various concrete coatings and sealers are available that will prevent or slow down the penetration of water and salts. Silanes in particular have been found to be very effective.

Concrete cover cannot protect dowels and tie bars because the opening at the joint interface allows for the inflow of chlorides, moisture, and oxygen. It is therefore common practice to protect dowels and tie bars from corrosion by using a protective barrier coating such as fusion bonded epoxy. The long-term effectiveness of epoxy coating of embedded steel is currently under intense study (Mehta and Monteiro 1993) although many cite it as an excellent preventive measure Perenchio (1994). Some agencies have investigated the use of plastic and stainless steel cladding as an alternative coating for dowel bars.

Recommended Approach for Prevention of MRD in Concrete Pavements

This section presents a summary of the approach adopted in the guideline provided in Volume 2 of this Final Report for the prevention of MRD in concrete pavements. The strategy taken applies the background information presented in the previous section to develop a more holistic approach, relying on the selection of constituent materials, proportioning, testing, and construction to ensure that concrete durability is achieved. The guideline concludes with specific recommendations provided for the mitigation of specific MRD types.

Constituent Materials Selection for Preventing MRD

The guidelines espouse the philosophy that quality concrete can only be produced from quality materials. Constituents are broken down into aggregates, cementitious materials, admixtures, and mixing water. An approach is provided to assist the engineer/mix designer in the selection of the various constituents. In the case of aggregate selection, a flowchart has been developed that presents a series of standardized tests that can be used to evaluate aggregate sources for all common aggregate-related MRDs. It is

recognized that many agencies have developed their own approaches for evaluating aggregates, commonly focusing on a single distress mechanism. For example, numerous procedures have recently been developed to evaluate the ASR susceptibility of an aggregate source, including the proposed AASHTO Guide Specification (AASHTO 2000) and the new Canadian Standards (Fournier et al 1999). In the guidelines, each agency is encouraged to evaluate its procedures and modify them as necessary to include methods that best address local conditions.

The first step in applying the flowchart is to evaluate the field performance of the aggregate source under study. If the field performance has been poor, it is recommended that the aggregate source be rejected until a thorough analysis can be conducted to determine what, if any role, the aggregate played in the noted failures. Good field performance does not immediately result in aggregate acceptance, as changes in other concrete constituents, mix design, or service conditions could result in deterioration in marginal aggregates that may not have shown up in previous applications. Instead, it is recommended that ASTM C 295 be conducted to characterize the aggregate source to identify whether the aggregate is potentially non-durable, based on susceptibility to AAR or freeze-thaw deterioration, or whether unstable minerals are present. If the aggregate is considered durable, it is approved for use in concrete. Potentially non-durable aggregates undergo testing as recommended. If it is concluded that the aggregate is indeed potentially non-durable in one or more categories, the guideline user is directed to either reject the source or mitigate according to recommendations provided.

Based on the aggregate selected, recommendations are made regarding the type of cementitious materials to be used. The selection of cement and supplementary pozzolanic/cementitious material is an extremely important element in designing durable concrete pavements. In many applications, the use of a standard ASTM C 150 Type I cement will provide satisfactory results, but this is not always the case. The proposed guideline details the selection of cements identified through ASTM C 150, *Standard Specification for Portland Cement*, and ASTM C 595M, *Standard Specification for Blended Hydraulic Cements*. Fly ash, GGBFS, and silica fume additives are also discussed with the emphasis on enhancing concrete durability.

Type I cement is most commonly used, although Type III cements are finding more widespread use in "fast-track" projects. In addition, ASTM C 595M provides standard specifications for blended hydraulic cements that feature an intimate blend of portland cement with fine materials such as GGBFS, fly ash or other pozzolans, hydrated lime, and pre-blended cement combinations of these materials (PCA 1992). Blended cements have not been commonly used on paving projects in the United States, but may offer some advantages in producing durable concrete pavements.

Some believe that the movement toward quicker setting cement and faster construction may be detrimental to long-term concrete durability. Mehta (1997) argues that for concrete to remain durable, it must remain relatively impermeable. He proposes that durable concrete should be slow-hardening with high creep and low elastic modulus at early ages to resist drying shrinkage and thermal cracking. Although he acknowledges that this can be accomplished through the use of coarser ground cement with relatively low C_3S content, he doesn't directly advocate a return to the cement of the past. He instead suggests that there is currently an overemphasis on speed of construction that has resulted in the unnecessary use of high early strength mixtures.

The proposed guidelines build on this theme, suggesting that instead of routinely using high cement content mixtures containing Type I or Type III portland cement, lower cement content mixtures containing blended cements should be considered. The lower heat of hydration and denser, less soluble microstructure will produce less permeable, more durable concrete. This would necessitate a major change in the way concrete quality is judged; possibly shifting strength requirements to 56 days or 90 days instead of the 28 days currently used. But, the use of slower setting concrete mixtures may be practical for many paving projects where speed is not paramount.

Cement requirements may also exist depending on the environment in which the pavement will serve. For example, if the pavement will be exposed to sulfates (a common impurity in deicing salts), a Type II or V



cement is recommended (ACI 1992a). For very severe exposure, a Type V with added pozzolan or slag with a demonstrated ability to improve sulfate resistance must be used. Additionally, ACI reports that reducing the permeability of the concrete through the use of a low w/cm and good curing practices is also needed to ensure good sulfate resistance. The inclusion of a low calcium pozzolan (e.g., Class F fly ash) or GGBFS also has a beneficial effect by reducing the permeability of the paste while minimizing the amount of soluble CH present. Further, the use of such supplemental cementitious materials can have a positive effect by mitigating ASR as is discussed later in the guideline. In general, it is evident that fly ash and GGBFS can be powerful and cost-effective allies in producing durable concrete through physical and chemical modification to its microstructure. The addition of low calcium fly ash or GGBFS will reduce concrete permeability while converting soluble CH into less soluble, more stable CSH. It is important to demonstrate the ability of the fly ash or GGBFS to mitigate ASR and increase sulfate resistance through standard tests such as ASTM C 227 and C 1293 for ASR and ASTM C 1012 for sulfate resistance.

Non-cementitious admixtures are commonly added to concrete mixtures during proportioning or mixing to enhance the freshly mixed and/or hardened properties of concrete. The guideline describes various admixtures that have been used in pavement construction in North America including air entrainers, water reducers, accelerators, retarders, corrosion inhibitors, non-cementitious minerals, and ASR inhibitors. The user of the guideline is directed to excellent descriptions of these various admixtures that can be found in a number of sources (PCA 1992; Mehta and Monteiro 1993; Mindess and Young 1981). All of these have been discussed in the previous section of this report. Obviously manufacturers' guidelines must be followed when using admixtures and testing must be completed on the job mix to avoid unexpected interactions between multiple admixtures. The final mix constituent considered in the guidelines is the mixing water. Recommendations for water quality are presented based on the standards contained in ASTM C 94.

Considerations in Proportioning and Mix Design for Prevention of MRD in Concrete Pavements

To this point, the guideline has considered the properties of the individual constituent materials of a concrete mixture. Once the materials are selected, they must be economically combined to form concrete that is workable and easy to consolidate, develops adequate strength, and has long-term durability. The PCA (1992) and the ACI (1991) both present acceptable methods for proportioning concrete mixtures. The discussion presented in the guideline focuses exclusively on elements of the proportioning and mix design process that have a direct bearing on enhancing concrete durability.

The key to durable concrete is the use of high quality materials arranged in a dense, relatively impermeable matrix. The selected aggregates should be strong and not be reactive or susceptible to frost or moisture damage within the environment in which they serve. If this condition is met, the hydrated cement paste is the weak link in the matrix, and thus the goal of proportioning aggregates is to ensure that they occupy as high a percentage of the concrete volume as possible. The hardened paste microstructure should be free of microcracking, possess a fine pore structure with an adequate air void system to protect it from freeze-thaw damage, and be composed of hydration products that are dense, stable, and relatively insoluble. Steel must be protected through adequate cover, and the concrete must be properly cured and free of drying shrinkage cracking. The guideline presents specific requirements for the selection of concrete mixture characteristics, aggregate grading, and tests for fresh and hardened concrete that have a bearing on concrete durability as discussed below.

As discussed previously, the w/cm is probably the single most important factor in this regard. It is recommended that the w/cm not exceed 0.45 for pavements that will experience freezing and thawing in a moist environment and will be exposed to deicer applications. In the same environment, reinforced concrete pavements should be constructed with a maximum w/cm of 0.40 unless an additional 12.5-mm of concrete cover is used (ACI 1991). Recommendations regarding exposure are presented in table 1-10 (ACI 1992a). It is generally not recommended that a w/cm exceeding 0.50 be used in pavement concrete.

As discussed in the Transportation Research Circular (TRB 1999), to ensure that an economical, high quality concrete mixture is obtained, the amount of cementitious material used should be minimized while maintaining the required w/cm . This can only be accomplished through a reduction in water content. Steps that can be used to achieve this outcome include the use of (TRB 1999):

- The stiffest practical mixture.
- The largest practical nominal aggregate size.
- The optimum ratio of fine to coarse aggregate.
- A uniform distribution of aggregate to minimize paste demand.

Once the w/c and cement content are selected, the physical characteristics of the aggregate must be considered. The two aggregate characteristics that have the largest impact on proportioning are the particle size distribution (i.e., gradation) and the nature of the particles (e.g., shape, porosity, and surface texture) (PCA 1992). These two parameters affect both the handling of fresh concrete and the properties of hardened concrete.

Traditional aggregate grading methods use a coarse aggregate and a fine aggregate as presented in ASTM C 33, *Standard Specification for Concrete Aggregates*. Both the ACI and PCA procedures recommend that the largest practical aggregate size be used for job conditions to maximize economy while minimizing void space and paste requirements. This is in contrast to recent pavement construction practice that has significantly decreased the maximum aggregate size, particularly in northern midwestern States. The use of smaller aggregate is in direct response to the inadequate freeze-thaw durability of some aggregate sources, but it has become standard practice in some States for all paving concrete, regardless of the specific aggregate's durability. It is recommended in the guidelines that this practice be reconsidered and that larger aggregate sizes be used if freeze-thaw durability is demonstrated. Blending of larger durable aggregates with smaller aggregates that have been crushed to enhance durability is on recommendation.

The use of a larger maximum aggregate size is one method to increase the proportion of aggregate in the mixture. A second method is to achieve a more continuous aggregate grading than is commonly used in traditional mix design methods. The U.S. Air Force has recently changed its specification for grading aggregates in concrete mixtures, considering both the coarseness and workability of the gradation (Muszynski et al. 1997). Aggregate is no longer considered to be either fine or coarse, but instead is treated as a single aggregate blend. Using these procedures, the Air Force believes that a more easily constructed, better performing concrete pavement is obtained. This approach is advocated in the recent Transportation Circular on concrete durability, which states that a lack of mid-sized aggregate (around 9.5 mm size) results in concrete with high shrinkage, high water demand, and poor workability (TRB 1999). Blending of multiple aggregate sources can be used to address this problem (Shilstone 1990).

It is believed that the use of the largest maximum aggregate size (considering freeze-thaw durability of the aggregate) with a more continuous grading may provide an opportunity to meet these goals. Additional research needs to be conducted on the constructability and performance of such mixes before significant changes to traditional proportioning schemes can be recommended.

Once the mixture proportions have been established, a number of standard tests are recommended that can be used to establish the susceptibility of concrete to MRD. Unfortunately, as discussed previously, many of these standards are designed to test only a single mix constituent, and not the mixture as a whole. Further, many of the tests are long term and are not well suited to mix design process. But even with these limitations, the guideline recommends the following standard test methods that may prove to be useful in assessing the potential durability of pavement concrete:

- AASHTO PP 34-99: *Proposed Standard Method of Test for Restrained Drying Shrinkage*.
- ASTM C 457: *Practice for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*.

- ASTM C 666: *Test Method for Resistance of Concrete to Rapid Freezing and Thawing.*
- ASTM C 672: *Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals.*
- ASTM C 1202: *Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration.*
- ASTM C 642: *Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete.*
- Proposed ASTM Sorptivity Test.

The guideline recognizes that the current suite of standard tests is limited and that research is currently under way to develop better predictive tests in the future.

Construction Considerations for Preventing MRD

Once a mixture design has been accepted, the next step is construction. Construction can have a very important impact on the occurrence of MRD in concrete pavements, and it is essential that the mixture be properly batched, mixed, consolidated, and cured under conditions that will produce little microcracking and stable hydration products. The guideline addresses the following construction practices that have a direct impact on concrete pavement durability:

- Ambient and seasonal conditions.
- Batching.
- Duration of the mixing cycle.
- Method of consolidation.
- Steel placement.
- Finishing.
- Curing.

The guideline discusses each of these topics, providing guidance that should be followed to ensure that construction does not negatively impact concrete pavement durability.

Controlling Specific Types of MRD

The final section of the guideline presents techniques that can be employed to prevent specific types of MRD in concrete pavements. An SHA that has had specific problems regarding one or more MRD types can use this section to examine what preventive strategies are available to avoid future occurrences. In addition, if the selection of constituent materials indicates that the aggregate source is ASR susceptible, the user of the guideline can turn to this section to find approaches to prevent ASR. This following provides a brief summary of the guidance provided in this section.

Paste Freeze-Thaw Deterioration

The only practical technique available to avoid paste freeze-thaw damage in a freeze-thaw environment is to entrain air voids of the proper size and spacing into the concrete matrix. Air-entraining admixtures are specified and tested under ASTM C 260 and C 233 and the air content of fresh concrete is determined using ASTM C 173 or C 231. It is noted that the measurement of overall air content does not separate entrapped air from entrained air, nor does it ensure that the air-void system in the hardened paste is adequate. Further, loss of air during slip form paving is not uncommon. For these reasons, if a SHA has been suffering paste freeze-thaw damage, the air-void system in hardened concrete should be assessed microscopically during mix design and as part of the construction process using procedures described in ASTM C 457.

Deicer Scaling/Deterioration

In general, deicer scaling is not a concern for high quality PCC that has been properly constructed. In general, it has been found that using a high cementitious content and a low w/cm (0.45 or less) will result in good scaling resistance (assuming proper finishing and curing). In addition, if fly ash or GGBFS are being used, a longer than normal curing period prior to deicer application must be implemented, favoring spring/early summer construction over fall. Testing using ASTM C 672 can be used to assess the deicer scaling potential of a concrete mixture.

Aggregate Freeze-Thaw Deterioration

If aggregate freeze-thaw deterioration is a potential problem as revealed either through experience and/or the results of constituent testing, preventive measures must be taken. The guideline states that the best method of preventing aggregate freeze-thaw deterioration is to reject the use of susceptible or marginal aggregates, and use only aggregates with demonstrated good field performance and/or laboratory testing results. But this is not a practical solution in many areas where sources of non-susceptible aggregate do not exist or where laboratory test results are not definitive.

Therefore, if a marginal aggregate source is being considered for use, it must undergo benefaction to reduce the susceptibility to this MRD. To this end, four potential methods are suggested to improve aggregate freeze-thaw resistance. The first method attempts to separate susceptible aggregate from non-susceptible aggregate using the aggregate bulk specific gravity. The second method is to blend the susceptible aggregate source with an aggregate source that is known to be frost-resistant. Blending simply dilutes susceptible aggregate with non-susceptible aggregate, increasing the overall freeze-thaw durability of the concrete. Unfortunately, this method is not considered to be very effective, as it has a tendency to simply delay the onset of damage rather than prevent it. The third method is to reduce the maximum aggregate size below the critical size needed to cause damage. This method has become standard practice in many States and overall has been effective in reducing the incidence of aggregate freeze-thaw deterioration. This practice, although seemingly effective in reducing aggregate freeze-thaw deterioration, has led to a significant increase in the paste requirement for pavement concrete and has compromised the structural integrity of cracks and joints relying on aggregate interlock for load transfer. The final approach is to combine the second and third, using a combination of blending and size reduction for aggregate benefaction. Aggregate sources with demonstrated susceptibility to freeze-thaw deterioration can be crushed to smaller, non-susceptible sizes, and then blended with larger aggregate obtained from a non-susceptible source. This will provide the best performance at minimal additional cost, as only larger size fractions need to be purchased and shipped.

Regardless of the benefaction method employed, the effectiveness must be tested in the laboratory prior to construction. It is believed that the test method must confine the aggregate in a concrete matrix, such as in ASTM C 666 (Method A) and ASTM C 671. The results of this testing should be correlated with a more rapid test such as AASHTO T-103 or ASTM C 295 to monitor consistency of the aggregate source or the Washington Hydraulic Fracture Test for use in construction control.

Alkali-Silica Reaction

In areas with known ASR problems or if testing of the aggregate indicates that ASR potential exists, preventive measures should be exercised to prevent ASR in the field. The guideline discusses two methods that have recently been put forward to address ASR problems: the proposed Canadian standard (Fournier et al. 1999) and the Proposed AASHTO Guide Specifications (AASHTO 2000). Both these methods have been discussed previously in this report, so they will not be reviewed again, but the emphasis is on eliminating or minimizing the use of ASR susceptible aggregate, limiting alkalis in the concrete, using pozzolans and/or GGBFS, or adding an ASR inhibiting admixture.

Regardless of the approach used, a variety of mitigation techniques are available to address ASR. One obvious method is to reject the aggregate source in favor of aggregate not containing reactive silica. Another option is to blend aggregate containing highly reactive silica with non-reactive aggregate, effectively reducing the total amount of reactive silica in the concrete. Other methods of aggregate benefaction include selective quarrying, heavy media separation, washing and scrubbing, and chemical treatment, all of which may have applicability in some cases (ACI 1998). If the potentially deleterious aggregate supply must be used for practical reasons, other mitigation techniques can be employed in combination with aggregate benefaction.

The most common approach to mitigation of ASR is to reduce the available alkalis in the pore solution. Typically, the use of low alkali cement is considered effective in reducing the total alkalis in the mixture, although using a high cement content may still result in ASR even if a low alkali cement is used. Other potential sources of alkalis include chemical admixtures, pozzolans, slag, aggregate, and mixing water, and the contribution of all of these must be summed with that from the cement to determine the total alkali content of the mix. In the proposed Canadian standard (Fournier et al. 1999), the total alkalis are limited to a maximum of 3.0 kg/m³ Na₂O_e for mild prevention to 2.2 kg/m³ and 1.7 kg/m³ Na₂O_e for moderate to strong preventative action, respectively. In addition to limiting alkalis, the use of other mitigation techniques may be necessary, including (Gress 1997):

- Use of a Type IP blended cement.
- Addition of high silica, low calcium fly ash (e.g., Class F fly ash).
- Addition of GGBFS.
- Addition of silica fume.
- Addition of a lithium compound.

The use of lithium-based additives is showing increasing promise. A number of lithium compounds are available for mitigation, with their effectiveness dependent on the compound used, addition rate, aggregate reactivity and cement alkalinity (TRB 1999). The proposed AASHTO Guide Specification (AASHTO 2000) provides considerable information on the use of various lithium compounds and recommended addition rates.

Regardless of the approach being used to mitigate ASR, it is desirable to test the reactivity of the mixture with all constituent materials combined according to the job mix formula. In the proposed AASHTO Guide Specifications, it is suggested that ASTM C 441, ASTM C 1260, and ASTM C 1293 be used to test mixtures containing all the constituent materials proportioned according to the job mix formula to verify mitigation. For each test, expansion criteria are set along with limitations. In the Canadian standards, it is recommended that a two-year test be run in accordance with ASTM C 1293 to fully assess the ability of slag and/or pozzolans to mitigate ASR. The guideline notes that research continues to be done to establish correlation between laboratory tests and field performance, but until correlation between laboratory tests and field performance are established, the best strategy might be to use local experience to develop mitigation strategies that are known to work.

Alkali–Carbonate Reactions

The surest way to prevent ACR is to screen aggregates prior to use, rejecting those that are potentially susceptible to ACR. If potentially reactive aggregates must be used, it is possible to blend them with non-reactive aggregates to limit the potential for deleterious ACR. The maximum size of the aggregate also affects its reactivity, with aggregate having a larger maximum size having the greater potential for deleterious reactivity (ACI 1998).

The use of low alkali cements is far less effective in controlling ACR than ASR, as even cement alkali content as low as 0.40 percent can still lead to initiation of this distress (Farny and Kosmatka 1997). The most important cement parameter to consider when potential ACR susceptibility exists is to minimize the

total alkali content per unit volume of concrete, and not focus exclusively on the alkali content of the cement (Ozol 1994).

Pozzolans are also not effective in controlling ACR. However, preliminary evidence suggests that lithium-based admixtures appear to control ACR although little research has been performed to support this finding. Thus the only method currently available to prevent ACR is to eliminate or significantly reduce that amount of reactive aggregate.

External Sulfate Attack

The guideline presents the need for high quality, impermeable concrete as a prerequisite for concrete being resistant to external sulfate resistance. Concrete with a low w/cm and high cement factor is consistently recommended, as it will have lower permeability and thus limit the amount of sulfate ions that can diffuse into the concrete to attack it. This requires good workmanship, curing, and a relatively rich mix with a low w/cm .

Many researchers have found a link between cement properties and resistance to external sulfate attack. ACI (1992) makes specific recommendations regarding not only the selection of the w/cm , but also the type of cement to be used in an aggressive sulfate environment. These recommendations were previously presented in table 1-10. In moderate to severe exposure levels, a Type II or V cement is required. Cement low in tricalcium aluminate (C_3A) should be used if sulfate attack is anticipated, and some researchers state that the aluminoferrite phase (C_4AF) of portland cement should also be limited. For this reason, the Type V cement has a maximum calculated C_3A content of 5 percent and a combined $C_4AF + 2C_3A$ content that does not exceed 25 percent. It is recommended that performance testing using ASTM C 452 and C 1012 should be considered to examine the sulfate resistance of portland cements and combinations of cements and pozzolans/slag, respectively.

For very severe exposure, a Type V with added pozzolan or slag having a demonstrated ability to improve sulfate resistance must be used. In these conditions, it is recommended that a Type VP or VS cement be considered. Supersulfated slag cements, if available, are also an option.

Class F fly ash is generally found to be beneficial to sulfate resistance, whereas Class C fly ash may actually be detrimental. For these reasons, high quality Class F fly ash should be considered most useful in improving sulfate resistance of concrete. It is thought that fly ash meeting ASTM C 618 and having less than 10 percent bulk CaO can be used to improve sulfate resistance. Fly ash containing 10 to 25 percent CaO should be tested with the actual materials to be used in the concrete. The replacement of portland cement with GGBFS also has beneficial effects toward sulfate resistance through the reduction of the C_3A content incurred by reducing the amount of portland cement in the concrete. It also reduces soluble CH in the formation of CSH, altering the environment required for the formation of ettringite. CSH also forms in pore spaces normally occupied by alkalies and calcium hydroxide, reducing the permeability of the paste.

Due to variability in the effectiveness of various techniques to improve sulfate resistance, it is important that specific combinations of the cement and pozzolan be tested to verify sulfate resistance. Unfortunately, assessing the sulfate resistance of concrete is difficult as there is currently no standard ASTM test for assessing the sulfate resistance of specified concrete made using the selected constituent materials and job mix formula. ASTM C 1012 is the most commonly recommended test to assess the sulfate resistance of portland cement, blends of portland cement with slags and fly ash, or blended hydraulic cements. But it only tests the resistance of the cementitious materials and not the concrete. Modifications to the standard test methods could be made so that the job-specific concrete is tested. This would be particularly useful as mix parameters are considered to be at least as influential as cement chemistry in the sulfate resistance of concrete.

Internal Sulfate Attack

The guideline recognizes that it is uncertain to what degree internal sulfate attack affects pavements and, as a result, it is difficult to make specific recommendations to prevent it. But it also recognizes that in light of some well-known premature pavement failures that have occurred in recent years, it seems prudent to consider internal sulfate attack as a potential problem in pavements. Prevention must focus on the evaluation of the concentration and chemical form of sulfur within constituent materials (cement, aggregate, and admixtures). Concrete permeability may also have an influence as a reflection of the ease of transport for various species within the concrete matrix. And avoiding excessively high curing temperatures is also necessary.

As with external sulfate attack, the composition of the cement is potentially an important factor for internal sulfate attack. Sulfate-related deterioration in concrete is generally related to the C_3A content of the cement and, to some degree, the C_4AF content as well. Gress (1997) stated that limiting the sulfate content of cement is an extremely important step in preventing potential DEF. To control DEF, he recommends that a maximum sulfate content for cement and cementitious materials be limited to no more than 3 percent, although he admits that this alone may be insufficient to eliminate deleterious DEF.

The Duggan Test has been proposed as a test method that provides a "rapid measure of the potential for chemical expansion in concrete" (AREA 1996). Expansion can be due to alkali–aggregate reactivity, internal sulfate attack, or other potentially deleterious reactions. As a result, petrographic information should be gathered to determine what deleterious chemical reaction is implicated in the expansion.

ASTM C 1038, *Expansion of Portland Cement Mortar Bars Stored in Water*, has been used to investigate the expansion of concrete resulting from calcium sulfate (gypsum) in the cement. It is applicable to the study of a specific portland cement and thus cannot be used to consider the influence of aggregate or admixtures.

The effect of fly ash or GGBFS on the occurrence of DEF is unknown. On the one hand, as discussed previously, pozzolans can increase concrete resistance to sulfate attack. But fly ash and GGBFS may also provide forms of sulfur that may lead to the adverse reaction by providing slowly soluble forms of sulfate. Another potential source of slowly soluble sulfate is the aggregate. Sulfates having very low solubility in water, such as barium and lead sulfate (which are common in some aggregates), are relatively harmless. But some aggregates may contain moderately soluble sources of sulfate that may be problematic. For instance, work published by Peterson et al (1999) found that calcium sulfide in slag coarse aggregate was soluble at pH levels common in concrete. A petrographic analysis of the aggregate, in accordance with ASTM C 295, should be used to identify potentially problematic sources of aggregate sulfates.

Aside from the properties of the constituent materials, the overall quality of the concrete may also affect its susceptibility to internal sulfate attack. Thus, recommendations made in the guidelines for the selection of quality materials, the design of a relatively impermeable mix, and the use of proper construction methods should help reduce the potential for internal sulfate attack. Specifically, using mixes with higher aggregate content and reduced heat of hydration while following recommended hot weather concrete placement practices should have a positive impact on the mitigation of internal sulfate attack of the concrete.

Corrosion of Embedded Steel

When considering the corrosion of embedded steel, the permeability of the concrete to chloride ion penetration is an important factor in determining whether the cover is adequate. This can be measured in the laboratory using ASTM C 1202, *Electrical Indication of Concrete's Ability to Resist Chloride Ion*

Penetration. The rapidity, ease of use, and reliability make this test very attractive when investigating the chloride ion permeability characteristics of concrete. Because of its usefulness, it is recommended that this test be routinely conducted as part of the mix design process as a tool to assess concrete permeability, as long as its limitations are recognized.

The guidelines recognize that even the most impermeable concrete can become permeable if cracking occurs. Thus, sound design practices are suggested to minimize cracking and to keep cracks that form in reinforced pavements tight, minimizing the infiltration of deleterious agents. Construction practices should be used that prevent plastic shrinkage cracking of the pavement surface through the application of an approved curing compound at the appropriate time. This is most important during dry, windy days when the evaporation of bleed water can be quite rapid. Poor concrete consolidation must also be avoided through the design of a workable mix and care in selecting the appropriate depth, spacing, and operating frequency for internal vibrators.

Concrete cover cannot protect dowels and tie bars because the joint interface allows for the inflow of chlorides, moisture, and oxygen. It is therefore common practice to protect dowels and tie bars from corrosion using a protective barrier coating such as fusion bonded epoxy. The long-term effectiveness of epoxy coating of embedded steel is currently under intense study although many cite its use as an excellent preventive strategy. Some States have investigated the use of plastic coatings and stainless steel cladding as alternative coatings for dowel bars.

Need for Record Keeping

The final section in the guideline strongly suggests that information collected prior to and during construction regarding the selection of materials, mixture design, and construction practices be collected and stored to assist in future investigations of MRD in concrete pavements. Due to the complexity of the deterioration mechanisms, and to some degree the lack of accurate data from in-service concrete pavements, there is some uncertainty regarding the prevention of many MRD types. SHAs are therefore encouraged to maintain detailed records consistent with ACI 126.1R, *Guide to a Recommended Format for the Identification of Concrete in a Materials Property Database*. This guide provides the required forms and examples of how to complete them. SHAs are highly encouraged to collect such data, as the use of a standardized data collection format by all SHAs will provide a unique opportunity for researchers to examine MRD occurrences over a wide geographical area. This will assist in answering some of the unknowns that currently exist in the understanding of deterioration mechanisms, ultimately leading to improvements in concrete pavement performance.

Summary of the Techniques for the Treatment, Rehabilitation, and Prevention of MRD in Concrete Pavements

This section of the final report described the background used to develop treatment, rehabilitation, and prevention strategies to address MRD in concrete pavements. Although a number of treatment and rehabilitation strategies are available, few can actually address the cause of deleterious MRD once it is under way. Most of the treatment strategies will simply delay the need for major rehabilitation, which commonly includes either the fracturing of the distress pavement slabs or overlaying or complete reconstruction. Thus, the most effective approach is to address MRD by preventing it through the selection of durable materials, proper proportioning and testing, and the application of sound construction techniques.

Chapter 4. Evaluation Of The Field And Laboratory Guidelines

4.1 Introduction

In order to assess the guidelines developed under this project, in-service concrete pavements exhibiting MRD were subjected to a coordinated series of field testing and laboratory evaluation activities. In this way, the usefulness and applicability of the guidelines could be evaluated under actual field conditions, thereby providing feedback on what areas of the guidelines worked well and what areas required modification or refinement.

This chapter briefly describes the field and laboratory testing activities that were conducted under this project. First, a discussion of the pavement project selection process is provided, including a listing of the six pavement projects selected for inclusion in the study. Next, a general summary of the data collection and analysis process is provided. Finally, a summary of the improvements to the guidelines that came out of the field and laboratory testing (and incorporated into the final version of the guidelines) is provided. Bound separately in Volume 3 of this Final Report are the individual case studies that include the condition survey, field sampling, and laboratory analysis results obtained for each of the projects, leading up to the identification of the most likely MRD type present in the pavement.

4.2 Project Selection Process

The goal of the project selection process was to identify candidate concrete pavement projects suffering from MRD so that the guidelines could be tested under "real-world" conditions. The project team asked members of the advisory panel as well as representatives from other highway agencies to submit candidate projects so that a significant number of potential projects would be available. Ideally, the candidate project will be exhibiting a heretofore unidentified distress so the ability of the guidelines to diagnose the distress could be evaluated. However, some level of diagnostic work had been conducted on several of the candidate projects.

The project team reviewed the list of candidate projects and prioritized the list based on the desire to have a range of MRD types included in the study and the need to have at least one suitable project in each of the four SHRP LTPP climatic regions (Wet-Freeze, Wet-Nonfreeze, Dry-Freeze, Dry-Nonfreeze). Although a total of four projects was originally specified for the evaluation of the guidelines, the project team decided to include some additional projects in order to provide a more complete assessment of the guidelines. To incorporate these additional projects without adversely affecting the budgetary limitations of the project, two sets of test sites were established:

- **Primary Test Sites**—Four test sites exhibiting MRD that are located in each SHRP LTPP climatic region. Each of these pavement test sites is subjected to the full battery of field and laboratory tests.
- **Secondary Test Sites**—Two test sites exhibiting MRD that provide an alternative site in a particular climatic region. These pavement test sites are subjected to the same level of field testing as the primary sites, but are subjected to less intensive laboratory testing.

The primary and secondary test sites included in the study are listed in table 1-12. Again, the primary test sites represent each of the four SHRP LTPP climatic regions, and also include a range of other factors including age, pavement design, and type of distress manifestation. The secondary sites are included to provide additional sections for assessing the guidelines. The general location of these projects is shown in figure 1-11.

Table 1-12. Primary and secondary test sites included in study.

State	Highway/City	Climatic Region	Pavement Design	Year Built
<i>Primary Sites</i>				
California	SR 58, Boron	Dry-Nonfreeze	230 mm JPCP 4.0-5.8-5.5-3.7-m joint spacing No dowels	1971
Minnesota	TH 65, Mora	Wet-Freeze	200 mm JPCP 4.0-4.6-5.2-04.6-m joint spacing No dowels	1989
North Carolina	I-440, Raleigh	Wet-Nonfreeze	200 mm JPCP 7.6-7.0-5.8-5.5 m joint spacing 32-mm dowels	1982
South Dakota	I-90, Spearfish	Dry-Freeze	200 mm JRPC 12.3-m joint spacing 25-mm dowels	1968
<i>Secondary Sites</i>				
California	SR 14, Mojave	Dry-Nonfreeze	215 mm JPCP 3.7-4.0-5.8-5.5-m joint spacing No dowels	1972
Iowa	SR 2, Nebraska City	Wet-Freeze	240 mm JPCP 6.1-m joint spacing 32-mm dowels	1986

4.3 Field Data Collection and Laboratory Testing

The pavement projects included in the study were subjected to a detailed field condition survey, and core samples were retrieved from prescribed locations for laboratory testing and evaluation. All of these activities were performed in accordance with the systematic procedures outlined in the guidelines presented in Volume 2 of this Final Report. This section briefly describes the field and laboratory testing activities that were conducted on the projects.

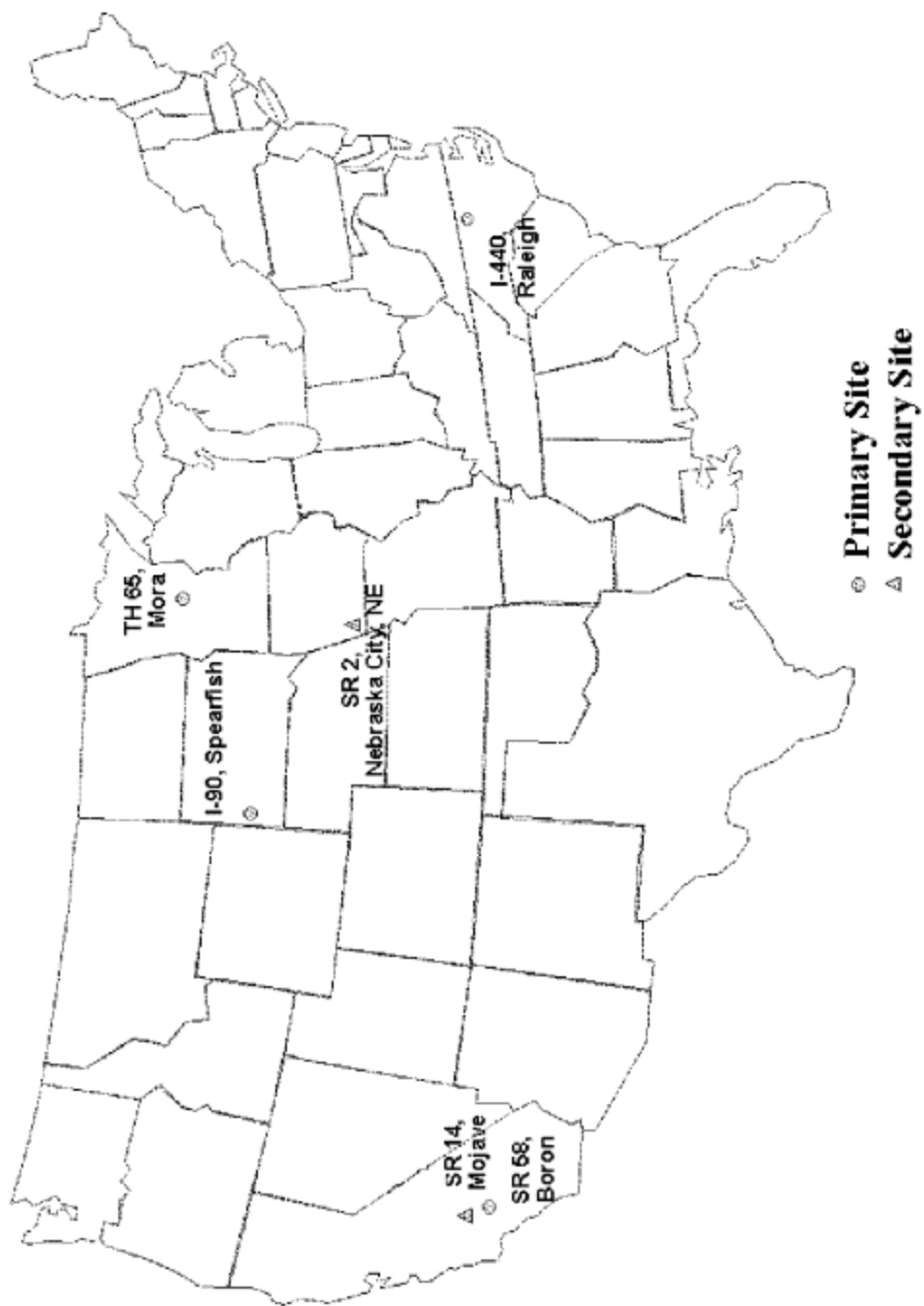


Figure 1-11: Graphic. General location of projects included in study.

Field Distress Data Collection

Preliminary Records Review

Prior to conducting the field evaluations, the project team solicited general design and construction information from the participating highway agencies for each pavement project. This included general location data, basic structural design data, specific mix design information, and traffic data. The limits of the project were defined as that length of pavement over which the structural design, date and method of construction, and paving and foundation materials are the same.

Survey Preparation

The project team made several preparations prior to the conduct of the field surveys in order to ensure that they proceeded as smoothly and efficiently as possible. These preparations included:

- Assembly of the field survey equipment (measuring wheel, faultmeter, paint, and so on).
- Preparation of the data collection forms, including:
 - General Information Form Overall Pavement Distress Survey Form
 - Overall Pavement Distress Summary Form
 - MRD Characterization Form
 - Typical MRD Survey Form
- Arrangement of coring and traffic control operations through the participating highway agency.

In addition, each project was assigned a unique project identifier, using the following naming convention:

[State]-[Highway Number]-[Beginning Milepost]-[Section Number]

where:

State = Two-letter state abbreviation (e.g., MN, IA).

Highway Number = Three-digit highway number (e.g., 015, 440).

Beginning Milepost = Three-digit number of lowest milepost of project limits (e.g., 220, 059).

Section Number = Three-digit number of section within the project (e.g., 001, 002).

Shoulder Survey

Prior setting up the traffic control and conducting the pavement distress surveys, the project team first drove the limits of the project to assess the overall pavement condition and the uniformity of distress levels, topography, and traffic volumes. In addition, candidate sections of the project for the field investigation were noted based on distress uniformity and traffic control and safety considerations.

During the shoulder survey, the project team drove along the shoulder at slow speeds (8 to 16 km/h) and noted the general pavement conditions by milepost or by distance as measured from the beginning of the project with the vehicle's odometer. In addition, any prevailing roadway characteristics that might have an effect on the performance of the pavement (e.g., areas of cuts/fills, topography, traffic volume/loading) were noted. These shoulder surveys were conducted in both directions to determine if any directional differences in performance or distress manifestations were apparent.

Section Layout

After the shoulder survey was conducted, the pavement sections to be included in the field surveys were selected. Generally, either one or two sections were selected within each project in accordance with the guidelines.

Once the section was selected and the traffic control established, a nominal length of 150 m was used in laying out the pavement sections to ensure consistency with the LTPP program. The actual section length depended on the joint spacing, but a minimum length of 150 m was always obtained for each section.

The section was laid out by selecting a beginning point (usually right at a milepost, station number, or other permanent marker or fixture to facilitate locating the sample unit in the future) and marking with paint the beginning of the section at the transverse joint nearest the marker. The beginning point was noted with a line and with a station number of "0+00" by defining this starting location as station 0+00, all stationing along the section was made relative to the beginning point.

Once the beginning point was established, a distance of 150 m was wheeled off in the direction of traffic to the end of the section. The end of the sample unit was marked at the far end of the slab in which the 150-m mark was encountered. While the sample unit length was being measured, additional marks and station numbers were made at 15-m intervals (e.g., 0+15, 0+30, 0+45, and so on) to aid in conducting the distress surveys. This stationing was also useful in noting the locations of samples retrieved, photos taken, and any other testing performed within the section.

Also during the section layout, members of the project team selected a representative slab within the section for core sampling. All of the core samples obtained from a given section to be used in the laboratory testing were obtained from a single slab and from the joints adjacent to the slab.

Pavement Distress Surveys

The distress surveys were performed in general accordance with the procedures and distress definitions found in the SHRP LTPP *Distress Identification Manual for the Long-Term Pavement Performance Project* (SHRP 1993). However, all of the cracking associated with the MRD was not recorded during the surveys; instead, the general area of MRD was noted on the modified LTPP forms and the physical manifestations characterized on the MRD survey form.

During the distress survey, all distresses occurring within the section were noted and, where appropriate, drawn approximately to scale using the modified LTPP distress survey form. To facilitate the recording of the various distresses, the distress map symbols were used. These were modified from the LTPP Distress Identification Manual, as attempts have been made to focus only on key distress types and also to reduce the distress survey to a *recording* of pavement distress as opposed to a *diagnosis* of pavement distress (e.g., the definition of D-cracking in the LTPP distress manual presupposes the identification of the MRD). The primary distresses that were evaluated during the distress surveys were:

- Corner breaks.
- Linear cracking (longitudinal and transverse).

- Joint seal damage (longitudinal and transverse).
- Joint spalling (longitudinal and transverse).
- Map cracking.
- Scaling.
- Polished aggregate.
- Popouts.
- Blowups.
- Faulting of transverse joints.
- Patch/patch deterioration.
- Water bleeding and pumping.

The survey crew consisted of two field engineers, one responsible for completing the data forms and one responsible for obtaining the following physical measurements:

- Joint faulting (measured for every joint at distances of 0.30 and 0.75 m from the outer slab edge).
- Joint widths (measured at every third to fifth joint, depending on the spacing).
- Lane-shoulder drop-off (measured every 15 m along the section).

The results of the distress survey were later tabulated in the office using the overall distress summary form.

The distress survey was generally conducted in the outer traffic lane of multi-lane facilities; the exception to this was on the South Dakota project, in which one of the sections was conducted in the inner lane because the observed distress manifestations were more prevalent.

MRD Characterization

After the completion of the distress survey, the survey team then conducted an evaluation of the MRD occurring on the section. The purpose of the MRD evaluation was not to identify the causes of the distress, but rather to characterize to the greatest extent possible the visible signs of the MRD. These visible signs include:

- Cracking pattern (location, orientation, extent, crack size).
- Staining (location, color).
- Exudate (presence, color, extent).
- Scaling (location, area, depth).
- Presence of vibrator trails.

The presence of these indicators on the pavement section was recorded by the survey team, as was the typical cracking pattern associated with the MRD on a "typical" slab.

Photo Documentation

At the conclusion of the distress surveys, a complete photo or video summary of the sample unit was conducted. The purpose of this photo summary was to document the performance of the pavement, as well as to record the prevailing foundation and drainage characteristics of the roadway. A standard set of photographs—shown in table 1-13—were taken for each project.

Table 1-13. Standard photographs taken for each project.

Photo Number	Depiction
1	General Information Form (for project identification)
2	Overview of project from section start
3–10	Typical pavement features (slab, transverse joint, longitudinal joint, shoulder joint, ditchline, visible drainage features)
11–20	Typical pavement distresses (spalling, cracking, MRD, and so on)
21	Overview of overall shoulder condition
22	Backshot of project from section end

Pavement Coring and Sampling

During the initial layout of the section, members of the project team selected a slab within the section for coring and sampling. The slab was selected based on its exhibiting levels of MRD typical of the entire section. All of the core samples for laboratory testing were obtained from this slab and its adjacent joints. A single slab was selected so that the coring and sampling operations could proceed quickly and rapidly in a concise area.

The number of core samples retrieved from the coring slab within each section depended upon the type and extent of the MRD that was exhibited as described in the guideline. If the MRD was concentrated at the joints, a minimum of four core samples was retrieved; if the MRD was distributed throughout the entire slab, a minimum of five core samples was retrieved.

Cores from both the transverse joints and the center portions of the slab were retrieved. The locations of the cores retrieved from pavements with MRD concentrated at the joints differed slightly from those retrieved from pavements with MRD distributed throughout the entire slab as described previously. The cores were designated with the letters A through E based on their location and purpose:

- Core A, joint core taken over a dowel—used to examine condition of dowel.
- Core B, joint core taken between dowels—used to examine concrete at joint interface.
- Core C, slab core taken 0.3 m away from joint and 0.3 m from slab edge—used to determine extent of deterioration away from joint.
- Core D, slab core taken in MRD-free area—used to determine possible presence of MRD below slab surface.
- Core E, slab core taken in MRD-infested area—used to examine type/extent of MRD.

Core Diameter

Generally, larger, 150-mm-diameter cores were preferred because they provide a larger specimen to work with in the laboratory. Given the heterogeneous nature of concrete and the desire to sample across joints, larger specimens were sought. However, 100-mm cores were still considered acceptable and, in several instances, 100-mm cores were retrieved because the participating highway agencies did not have 150-mm core barrels. All core samples were taken through the entire thickness of the concrete.

Specimen Retrieval

The location of the cores was marked on the pavement and a directional arrow drawn on the surface to indicate the direction of traffic. The location of each core within the section was noted using the relative stationing system. The coring crews from the participating highway agencies then pulled the cores, after

which the project team dried them, marked them, and made a cursory visual examination. Joint cores that separated at the joint interface were rematched and secured with duct tape.

To aid in the identification of the core, an identification label was prepared and affixed to each core sample. A unique identifier was assigned to each core using the following naming convention:

[State]-[Highway Number]-[Beginning Milepost]-[Section Number]-[Core ID]

where:

State = The two-letter state abbreviation (e.g., MN, IA).

Highway Number = The three-digit highway number (e.g., 015, 440).

Beginning Milepost = The three-digit number of lowest milepost of project limits (e.g., 220, 059).

Section Number = The three-digit number of section within the project (e.g., 001,002).

Core ID = The one-letter code designation indicating core location (e.g., A, B).

A coring log sheet was also kept by the project team during the coring operations to completely document the location, manner of retrieval, and field condition of all cores.

Packaging and Shipping of Cores

After the core was marked and labeled, it was carefully wrapped in plastic bubble wrap packing material and placed in a shipping container for transport back to the laboratory. For specimens that were transported to the laboratory by common carrier, special care was taken protect the cores from damage by limiting the number of cores placed in a container and providing additional packaging material.

Laboratory Analysis

The laboratory analysis was conducted as previously described and as presented in the guideline in Volume 2 of this Final Report. In the course of this study, all methods discussed were utilized except for XRD.

The sample of concrete exhibiting distress came into the laboratory and, after being logged in, was first visually inspected. Specimens were produced from the core sample for use in the stereo OM for initial optical analysis and staining techniques to help identify ASR or sulfate phases. Next, thin sections were commonly produced and viewed in the petrographic microscope and/or SEM. This process of using the stereo OM, petrographic OM, and SEM is iterative and it is quite common to view the same specimen in all three instruments. Staining in particular can assist in the optical evaluation although it may interfere with SEM analysis.

When analyzing a concrete specimen, the concrete is viewed as a system of four principal components: air, hydrated cement paste, coarse aggregate, and fine aggregate. All available methods to examine the system and its components should be used, looking for all features that will help in the diagnosis. The ability to establish certain features as being normal greatly helps in deducing the cause of the problem. For example, no apparent coarse aggregate cracking all but eliminates aggregate freeze-thaw deterioration as a cause. As another example, the presence of an adequate, uncompromised air-void system helps rule out paste freeze-thaw damage as the primary distress mechanism. Systematic examination of all components of the concrete is crucial to determining the cause of failure.

The laboratory data sheets were used throughout the course of this study. One copy of the completed laboratory data sheets, along with sampling and field data sheets, accompanied each core through the laboratory evaluation. Laboratory personnel completed only those data sheets that applied to the analyses conducted.

Data Interpretation

The procedure for interpreting the data for this study used the flowcharts and tables presented in the guideline in Volume 3 of this Final Report. These flowcharts are used to guide the analyst toward diagnosis. It was found that the best results were obtained when the analysts continued through all the flowcharts before going back to confirm the first MRD identified. This accomplished two things. First, it helps remove the tunnel vision of looking at only one possible MRD. Second, moving through the flowchart will identify questions that require additional techniques to answer. Overall, this process helps to identify: 1) What questions need to be answered? 2) What techniques will be required? and 3) What samples will be needed? After completing the required analyses, the analysts would go through the flowcharts one more time, isolating the most probable MRD(s).

Chapter 5. Summary And Conclusions

This *Volume 1: Final Report* for the project entitled, "Detection, Analysis, and Treatment of MRD in Concrete Pavements," presented a synthesis of relevant information and a description of how this information was used to develop a set of three guidelines. It then summarizes the results of a total of six case studies in which the guidelines were applied. It was found that the first two guidelines were effective in making reasonable diagnoses of MRD, providing a valuable structure for both the engineer and petrographer to operate within.

The synthesis discusses the various deterioration mechanisms that result in MRD in concrete pavement. These distresses can generally be classified as either physical or chemical processes. Freezing of concrete in a saturated or near saturated state can result in freeze-thaw damage to the paste, coarse aggregate, or both. The application of chemical deicers can magnify the physical distress mechanism and, in some cases, deicers may react chemically to the detriment of the concrete structure. Common chemical attack mechanisms can be associated with the paste or an interaction between the paste and aggregate. Alkali-aggregate reactions, external sulfate attack, internal sulfate attack, and corrosion of embedded steel are chemical distress mechanisms. For each MRD, the distress mechanism, manifestation, diagnosis, treatment, and prevention are discussed.

The synthesis reveals that although some MRD types are well understood, there are many unanswered questions regarding the specifics of the deterioration mechanisms. Typically, multiple theories exist, none of which seems to completely explain laboratory observations. In addition, more than one distress mechanism is typically at work at the same time. The primary mechanism begins the deterioration process by producing microcracking in the paste, which in turn opens up the structure to the ingress of other deleterious substances. For example, it is common to find ASR and DEF acting together, but it is difficult to determine which mechanism initiated the distress. Similarly, it has been speculated that SEF in some instances fills pore void space to the point where the air-void system is compromised, resulting in paste freeze-thaw deterioration. This complicity of more than one mechanism makes it very difficult to determine the exact cause of deterioration, which in turn complicates the selection of appropriate treatment and prevention strategies.

The visual manifestations on the pavement surface of many of these deterioration mechanisms appear similar, especially early in their development. Cracking and staining in the vicinity of joints is an indicator of MRD, but this visual analysis alone does not provide positive identification of what mechanism is at work. For example, it is believed that in the past some cases of external sulfate attack, possibly resulting from deicer impurities, may have been misdiagnosed as aggregate freeze-thaw deterioration (D-cracking). It has also been speculated that DEF has commonly been misdiagnosed as ASR, even when conventional laboratory procedures have been used (Wolter 1997).

To address the difficulties of accurate diagnosis, standardize diagnostic methods executed by well-trained personnel are required. This includes training of both the field crews collecting visual information and concrete samples, and the laboratory staff carrying out chemical and petrographic analyses. In some cases, advanced analytical methods based on the use of SEM and XRD may be required to establish what mechanism(s) is at work. And it must be fully understood that the complexity of the problem might be so great that the best result of a diagnostic investigation is a prioritized list of probable causes.

The major component of this study was development, evaluation, and refinement of the first and second guidelines. To accomplish the evaluation, six test sites were chosen with at least one being in each LTTP climate zone. The guidelines, provided in Volume 2 of this Final Report, were applied at the six sites with the cooperation and support of the SHAs. Both *Guideline I – Field Distress Survey, Sampling, and Sample Handling Procedures for Distressed Concrete Pavements* and *Guideline II – Laboratory Testing, Data Analysis, and Interpretation Procedures for Distressed Concrete Pavements* were evaluated. The third guideline, *Guideline III – Treatment, Rehabilitation, and Prevention of Materials-Related Distress in Concrete Pavements*, was only superficially applied since detailed information regarding agency

maintenance, rehabilitation, and mixture design policy were unknown. In general, the guidelines seemed to direct the necessary work well and provide a systematic method of gathering and recording data. A detailed description of the case studies is provided in Volume 3 of this Final Report.

The first guideline presents a systematic approach for performing a field distress survey, sampling the distressed pavement, and sample handling procedures. The procedures were applied and few modifications were required from the draft guidelines. The only significant modifications were changes to the forms used for recording construction records data, which was done to better reflect the data that were available on the sites examined. One key point learned from applying the guidelines was that construction records for the selected sites were incomplete or limited. In part this may be due to the age of the pavements. However, it may be indicative of a systemic lack of methods and procedures for accurately recording construction data. Even data as fundamental as the job mix formula for the mix design was unavailable in many cases. Information such as climatic conditions during placement is non-existent. Improving data collection, most probably by an automated data collection system during concrete placement, could greatly add to the information available to help diagnose the causes of pavement distress, including MRD. It is suggested in Guideline III that SHAs adopt a more rigorous data collection and storage methodology in line with what is presented in ACI 126.1R, *Guide to a Recommended Format for the Identification of Concrete in a Materials Property Database*.

Applying Guideline I provides an engineer with a detailed assessment of the current condition of the pavement. This not only provides the current information needed for analysis, but also provides a baseline for monitoring the rate of pavement deterioration when compared to data gathered in the future. This greatly improves the ability of the engineer to maintain the pavement and extend its life, while providing a means to judge the effectiveness of various treatments. This also illustrates one positive aspect of the developed guidelines. They are intended to be applied together but they can easily be applied in different time frames. As an approach, a SHA may use Guideline I as a means of screening pavements and prioritizing maintenance and reconstruction after the specific MRD(s) present has been identified.

Of the three guidelines developed, the second guideline is the heart of the research effort, as it proposes an approach to diagnosing MRD. The recommended laboratory procedures provide a systematic method for analyzing distressed concrete. The diagnostic flowcharts and tables provide a step-by-step approach to use when trying to determine the exact cause of MRD. Clearly, there will be cases where the guidelines fail to isolate the cause to any one MRD mechanism and, in many cases, multiple MRD mechanisms will be identified as possible contributors to the observed distress. However, it is believed that in most cases the data collected using the methods discussed in Guideline II will provide a more complete understanding of the distress. Based upon the results of this evaluation, the majority of cases can be resolved. In four of the six case studies used to evaluate the guidelines, definitive and most probable causes of MRD were established. Of the other two, one was identified through execution of the guidelines as not likely being affected by an MRD. It is noted that the laboratory investigation conducted on this site bore out this conclusion, even though early stages of MRD were observed, but were not yet (and may never be) associated with pavement deterioration. In the last site presented, a diagnosis could not be reached using the guidelines, as it became evident early in the evaluation that a different approach would need to be taken to investigate the problem.

The researchers were satisfied with Guideline II in terms of its efficacy and broad applicability. However, in a couple instances, techniques not originally proposed were employed. Specific examples are the use of epifluorescence microscopy as a means of estimating the effective w/c for the concrete and the use of a flat bed scanner as a low cost imaging tool. Neither of these techniques is precluded by the guidelines. In fact, in the case of epifluorescence microscopy, it is presented in the synthesis. The methods proposed in the guidelines were never intended to be the sole methods of analysis or interpretation. They are simply designed to provide guidance as to the common methods. In the case of the flat bed scanner, this technology was used to show SHA personnel a low cost alternative to conventional microscopy. The flatbed scanner will not supplant the petrographic or stereo optical microscope, but it may provide an economical way for a SHA to begin the process of laboratory analysis.

by providing a means of documenting specimens so specific areas can be identified for analysis when sending specimens to an outside lab. When combined with staining techniques, the flat bed scanner is a useful addition to any concrete analysis laboratory.

For engineers working on this project, Guideline II proved to be very useful for helping them understand the process of laboratory analysis. For many engineers, this process is a mystery and misunderstandings can result if the procedures used are not understood by the person interpreting or otherwise using the data—the engineer. When following the guideline, the choice of tests was understood and the engineers knew that the laboratory personnel progressed through the diagnosis without stopping at the first distress identified. It is important when diagnosing MRD that the analyst keeps investigating all possibilities to discover all mechanisms that may be active.

Unfortunately, for laboratory personnel familiar with the various analytical techniques, the guidelines were reported to be too confining. Laboratory personnel examining concrete are, in general, slow to rush to judgment. The inherent variability in concrete, and the limited sampling possible from most pavements, makes it difficult for an analyst to make yes/no decisions about observations, as is required in the diagnostic flowcharts presented. Laboratory personnel are more comfortable with decisions that are not absolute or are somehow weighted for their significance. As the recipient of the data, the engineer has to understand that absolute decisions are rare and that in the end, the petrographer or analyst can only provide their best judgment. However, the laboratory personnel need to understand the engineers' needs. Namely, they need to make yes/no decisions about replacement or rehabilitation and, therefore, require the clearest possible diagnosis from the laboratory in order to proceed. Performing the laboratory analysis in accordance with Guideline II removes ambiguity and provides a comprehensive look at all possible distresses.

A strong point of Guideline II is that it does not force the diagnosis to resolve at one specific cause. Numerous MRD mechanisms can be active and each should be clearly identified, without bias. The guidelines serve as an interface between engineers and laboratory personnel. Although some MRDs will not be unambiguously diagnosed by using the guidelines, the more common distresses will be identified. Even when absolute diagnosis is not possible, the guidelines help the engineer understand the likely possibilities and the tests available to diagnose the problem further by contracting with outside laboratories.

With regards to Guideline III, the review of the literature suggests that the various strategies used to treat pavements affected by MRD are not very effective. Most treatments are short-term fixes, such as the application of surface sealers in an attempt to slow the ingress of moisture and deleterious compounds. Some suggested treatments, such as the use of lithium salts in treating ASR, show promise. But in general, long-term treatment of a pavement seriously affected by MRD almost always requires major rehabilitation, either through rubblization and overlaying or complete reconstruction.

Thus, the best method to treat MRD is to prevent it. In new construction, it is recommended that a holistic approach be adopted, in which the overall quality of the concrete is emphasized. Long-term strength is only one measure of quality and it is important that permeability also be considered. It is believed that 28-day strength is not the best measure of concrete quality and, in fact, the use of such short-term strength testing may be complicit in the increased observation of MRD. Instead, the emphasis should be shifted to producing dense, impermeable concrete having relatively defect-free insoluble paste microstructural characteristics. This requires the use of durable, non-reactive aggregates arranged to minimize the paste fraction. The paste should have low permeability and solubility. The use of high quality fly ash or GGBFS may offer advantages in achieving the desired concrete properties. And care must be exercised during all phases of construction to ensure that the concrete reaches its full potential.

To construct truly durable concrete pavements, it is believed that SHA incentives will need to be modified by changing construction specifications and practices to focus on long-term durability, de-emphasizing rapid construction and short-term strength gain unless project constraints absolutely demand "fast track" construction. A number of studies have suggested that the movement toward increased construction

speed has sacrificed concrete quality. This has included the use of ever shorter mixing times, increased speed of paving, high frequency vibration, and poor curing practices, all of which may have a negative effect on concrete durability. The general consensus is that the knowledge exists to build high quality, durable concrete pavements, but that the industry incentive is biased toward rapid construction and early opening. The best way to increase long-term durability of concrete pavements is to revise the incentive away from rewarding rapid construction (which uses short-term 28-day strength as a primary criteria) toward rewarding long-term concrete properties such as 90-day strength and permeability. This must be accompanied by an increased emphasis on selecting high quality, durable materials. It is realized that this will lead to an increase in initial costs, putting concrete pavements at a competitive disadvantage if life cycle costing is not considered. Therefore, the revision of highway agency policies must not be restricted to the engineering level, but also must include a commitment to accept higher initial costs to achieve high performance, durable concrete pavements that will provide many years of maintenance-free service. Without this commitment, it is unlikely that proposed changes can be implemented.

Appendix A: Distress Mechanisms

A.1 Deterioration Due to Physical Mechanisms

Physical attack mechanisms are related either to cyclic freezing and thawing of the concrete and/or the application of chemical deicers to keep the pavement surface free of ice. The three types of physical attack mechanisms considered here are freeze-thaw deterioration of the paste, deicer scaling/deterioration, and freeze-thaw deterioration of aggregate.

Freeze-Thaw Deterioration of Paste

When moist concrete is exposed to alternating cycles of freezing and thawing, internal deterioration can result. As this deterioration accumulates, it is referred to as freeze-thaw damage. Damage related to freezing and thawing can occur in both the cement paste and aggregate phases of concrete. It has long been recognized (Powers 1945) that damage to the cement paste phase can occur internally or at the surface. The issue of surface scaling is addressed later in the discussion of deicer scaling/deterioration, whereas susceptibility of aggregates to freeze-thaw damage is described in the section on aggregate freeze-thaw deterioration. This section focuses exclusively on the damage that can occur to the cement paste phase internally due to repeated cycles of freezing and thawing.

Paste freeze-thaw damage in concrete is considered to be a physical phenomenon arising from excess internal pressures resulting from the freezing action of water. Currently, a general consensus on the exact mechanism(s) responsible for the generation of these excessive internal pressures does not exist. The most widely accepted theories consider either hydraulic or osmotic pressures (or a combination of the two) to be the primary causes. The role of entrained air voids is a central element in protecting the paste from damage in both of the theories. It is generally agreed that the magnitude of these internal pressures is dependent on the concrete pore structure, moisture content, pore water chemistry, rate of freezing, and/or length of the freezing cycle.

The temperature at which water will freeze is a function of the size of the pore in which it is contained and the concentration of dissolved species in the water. An excellent review of the literature related to these phenomena is provided by Marchand et al. (1994). Because of physical and chemical surface interactions between water and the surfaces of hydrated cement, the smaller the pore size, the lower the temperature required to cause freezing. Because of their small size, the water held in the gel pores will not freeze at the temperatures to which concrete is exposed in a natural environment. Because the pore water solution in a hydrated cement paste contains varying levels of sodium and potassium alkalis, the water in the capillary pores tends to become supercooled (in which it remains in a liquid state at temperatures well below 0°C) before freezing. This effect is magnified if deicing salts are also present.

Powers (1945) first attributed freeze-thaw damage to excessive hydraulic pressures resulting from the expansion of ice. It was proposed that as ice gradually forms at discrete sites in a saturated capillary pore system, the resulting 9 percent volume expansion causes the surrounding unfrozen water to be expelled away from the freezing sites. Depending on the nature of the pore system, excessive internal stresses can be generated by hydraulic pressures resulting from resistance to this flow. The pressurized water moving away from the freezing sites would find relief at the air voids, where it could then presumably freeze without causing damage.

Based on experimental results, Powers recognized that the spacing between voids, rather than the total volume of air, was the better measure of resistance to freeze-thaw damage. Building on this, he developed equations that provided an average measure of the distance that water must travel to reach the surface of an air void (Powers 1949). He proposed the adoption of a void-spacing factor, now known as the Powers spacing factor, as the basis of protecting the paste from freeze-thaw damage. It is interesting to note that this pioneering work based on the hydraulic pressure theory still forms the primary basis of specifying freeze-thaw resistant concrete (see ASTM C 457).



More recent theories (Powers 1975) consider osmotic potential to be the primary cause of excess internal stresses. As previously mentioned the temperature at which water will freeze in concrete is a function of the alkali concentration as well as the pore size in which it is contained. Freezing will only occur when the temperature becomes low enough to allow ice to form at the existing alkali concentration. Because of their relatively large size, air voids are likely initial freezing sites and as the pore water solution freezes, only pure water forms the ice. Thus, the remaining unfrozen liquid at the freezing sites becomes a more concentrated alkaline solution. The less concentrated alkaline solution in the surrounding paste is then drawn to the freezing sites to maintain thermodynamic equilibrium. The driving force for the movement of this solution is a function of the alkali concentration gradient. As the unfrozen solution at the freezing sites is diluted by the infusion of surrounding water, additional ice growth occurs. This progressive ice formation can occur at any solute concentration, including zero and is referred to as ice-accretion (Powers 1975). Recent experimental studies have provided some support for this mechanism (Wang et al. 1996).

This process of pore water moving from the capillary system to contiguous air voids will continue until one of two possible conditions prevails. If adequate air-void space exists, sufficiently distributed throughout the paste, all of the freezable water will eventually diffuse to the freezing sites inside the air voids, eliminating any further fluid flow. This is desirable, since the resulting total absence of freezable water in the surrounding capillary pore system eliminates the possibility of paste frost damage. The other possible outcome is that the air void space is inadequate to accommodate all of the surrounding unfrozen water. If this occurs, osmotic pressures will increase due to the remaining differences in alkali concentrations between the solution surrounding the ice-filled voids and the bulk solution within the capillary pores. Pressures of any kind, whether caused by loads, hydraulic forces, or osmotic forces, that approach or exceed the tensile strength of the hardened cement paste will naturally cause damage. Also, if the air-void volume is inadequate, freezable water will remain in the capillary pores. This freezable water in the capillary pore system is susceptible to ice crystal growth at sufficiently low temperatures. Although extreme temperatures are required to freeze water entrapped in capillary porosity, excessive hydraulic pressures may then result as unfrozen water is forced through the pore structure due to ice expansion. Also, if the rate of temperature drop is too fast to allow all of the water to diffuse to the air voids, damage may occur.

Deicer Scaling/Deterioration

Deicer Scaling/deterioration is typically characterized by scaling or crazing of the slab surface due to the repeated application of deicing chemicals. Although the exact causes of deicer scaling are not known, it is commonly believed to be primarily a physical attack. But some recent studies suggest that chemical degradation of the cement paste may also be occurring, resulting in dissolution of calcium hydroxide, coarsening of the concrete pore system, and potentially the formation of deleterious compounds. Both the physical and chemical deterioration mechanisms are discussed below.

As mentioned, while the exact causes of deicer scaling are not known, it is commonly thought to be primarily a form of physical attack, possibly resulting from a combination of factors (Mindess and Young 1981; ACI 1992a; Marchand et al. 1994; Pigeon 1994; Pigeon and Plateau 1995). Some researchers have stated that the presence of deicing chemicals, particularly salts, can magnify or amplify the same mechanisms that lead to freeze-thaw deterioration of the paste as discussed previously (e.g., hydraulic and osmotic pressures) (Ghafoori and Mathis 1997). Contributing to this effect is the fact that pore water containing a relatively small amount of dissolved salts is more easily absorbed into capillary pores, resulting in increased saturation of the concrete (Pigeon and Plateau 1995, Ghafoori and Mathis 1997).

In addition to the hydraulic and osmotic pressure theories discussed in the paste freeze-thaw section, another physical theory of why the application of deicing salts can produce concrete scaling is that of salt crystallization within the pores (Hansen 1963). It is hypothesized that the pore solution can become supersaturated as wetting and drying cycles concentrate salts in the pores (Harnik et al. 1980; Mindess

and Young 1981). Once crystallization begins, salt molecules are drawn out of smaller pores into the larger pores, inducing potentially harmful crystallization pressures (Ghafoori and Mathis 1997).

It is apparent from the literature review that the physical mechanisms of deicer scaling are not completely understood. As stated by Pigeon (1994), deicer “scaling is a much more complex problem than frost induced internal cracking for many reasons, but probably in good part because it is related to the microstructure of the very surface layer or ‘skin’ of concrete.” Little is really known about the microstructure of this “skin,” except that it is composed predominantly of paste and that it is easily affected by poor finishing or curing practices. Because of this, it is subjected to high humidity gradients, drying, and microcracking and possibly an altered pore structure that is more susceptible to freeze-thaw damage (Pigeon 1994).

Although most research suggests that salt scaling is primarily a physical deterioration mechanism, some researchers have suggested that more attention should be paid to the chemical interaction between the deicing salts and cement paste hydration products (Marchand et al. 1994). Recent research indicates that the chemical interaction between deicers and concrete may not be as wholly benign as much past research suggests.

One major chemical degradation mechanism resulting from the long-term application of the popular chemical deicer sodium chloride (NaCl) is the dissolution of calcium hydroxide (Ca(OH)₂). The dissolution equation is stated as follows (Marchand et al. 1994):



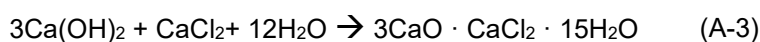
It is suggested that the dissolution process results in increased porosity at exposed surfaces, increasing the permeability of the concrete. This in turn increases the amount of water available and that which will freeze at 0°C due to increased pore size (Marchand et al 1994). One recent study conducted by the Michigan Department of Transportation (MDOT) adds credence to this theory. Muethel (1997) confirmed through laboratory analysis that depletion of calcium hydroxide led to an increase in permeability and reduced alkalinity of the concrete in the vicinity of cracks and joints. The reduction in the concrete pH contributed to corrosion of reinforcing mesh in the jointed reinforced pavements.

A subsequent reaction may occur in which the soluble calcium chloride reacts with aluminate phases in the paste to create chloroaluminate crystals according to the following equation (Marchand et al 1994):



It appears that this reaction would be expansive, although no citation was found in the literature regarding the expansive pressures that would be exerted, or whether it would be damaging in and of itself. Buck (1985) mentions that chloroaluminate preferentially replaces ettringite when salt is present. Buck also discusses that chloroaluminate formed during initial hydration in the presence of calcium chloride could later convert to ettringite if the solution becomes rich in sulfate ions.

Collepari et al. (1994) cited a number of studies that have concluded that CaCl₂, another common deicer, is associated with a deleterious chemical reaction with concrete. The chemical attack is accompanied by the formation of a hydrated calcium oxychloride according to the following reaction:



The reaction proceeds most efficiently at temperatures just above freezing (5°C to 10°C), with rapid formation of hydrated calcium oxychloride. This reaction is considered to be disruptive to the concrete matrix because of the hydraulic pressures generated. Collepardi et al. (1994) speculate that the damaging nature of this reaction has been masked by corrosion of reinforcing steel and freeze-thaw deterioration to the paste, but state that the chemical degradation that occurs is very detrimental. Collepardi et al. (1994) cite experimental evidence, based on decreasing compressive strength, suggesting that severe deterioration occurred in non-air entrained concrete exposed to CaCl₂ deicers even though there was no steel to corrode nor was the concrete subjected to temperatures below freezing.

It would be expected that factors leading to an increased permeability would be more prevalent at exposed concrete surfaces. In addition to contributing to surface scaling, concrete faces along joint or crack walls would also be adversely affected. A number of studies appear to support this hypothesis. The previously cited study conducted by MDOT observed that CH leaching was pronounced in concrete cores obtained at transverse crack locations in jointed reinforced concrete pavements from three different State routes (Muethel 1997). Visual surveys of the concrete pavement surface revealed localized deterioration and staining, while laboratory analysis confirmed depletion of CH, increased permeability, and reduced alkalinity of the concrete.

A study conducted for the Ohio Department of Transportation (ODOT) investigated the joint deterioration observed in many northern tier States (Munoz and Chou 1994; 1996). Termed "coning," this deterioration is characterized by concrete degradation at joints that progresses from the bottom of the slab upward. The exact cause of this deterioration is unknown, but it is speculated that more than one distress mechanism might be at work. Munoz and Chou state that dowel baskets may interfere with proper concrete consolidation, resulting in weaker concrete and increased porosity and permeability. This contributes to a chemical attack mechanism, with dissolution of the concrete considered the primary culprit. Munoz and Chou (1994; 1996) explored the dissolution mechanism, stating that both the presence of dissolved salts and the velocity of solvent flow through the concrete are important factors. It is speculated that concrete at the joints has increased permeability, with accompanying relatively high solvent velocity. Additionally, potentially high CO₂ content in the solution from melted snow and other factors may lead to an increased solubility of the concrete.

Another potential detrimental effect of the application of chemical deicing salts is increased alkali-silica reactivity. Nixon and Page (1987) report that salt contamination of concrete, whether through the incorporated aggregate or application of deicing salts, "could be expected to increase the alkalinity of the pore solution and hence the likelihood of damaging alkali silica reaction." The mechanism cited is the combination of sodium chloride with calcium hydroxide and tricalcium aluminate, precipitating calcium chloroaluminate or chloro-sulphoaluminate gel as previously described. They state that in cases where sodium chloride is made available through deicer applications, the effect is to produce a zone of reduced alkalinity near the exposed concrete surface, an occurrence consistent with the observations of Muethel (1997).

Aggregate Freeze-Thaw Deterioration

Freeze-thaw deterioration of aggregate is associated with the freezing and thawing of susceptible, coarse aggregate particles in the concrete. The resulting pavement distress is commonly referred to as D-cracking. Aggregates identified as being freeze-thaw susceptible either fracture as they freeze and then dilate resulting in cracking of the surrounding mortar, or they allow for rapid expulsion of water during freezing contributing to dissolution of soluble paste components at the aggregate-paste interface. Key aggregate properties related to freeze-thaw susceptibility are aggregate size, pore size distribution, and strength (Mindess and Young 1981). Most susceptible aggregates are of sedimentary origin (e.g., cherts, sandstones, shales, limestones), can be calcareous or siliceous, and can be gravel or crushed rock (Neville 1996).

Two different mechanisms are associated with aggregate related freeze-thaw damage, including damage to the aggregates themselves and/or damage to the adjacent paste system. Powers' hydraulic pressure theory is generally considered to provide a reasonable description of the actions taking place inside aggregate particles during freezing (Pigeon and Plateau 1995). In a critically saturated aggregate, excessive pressures can develop due to the volume increase associated with ice formation. The expansion associated with ice formation may be accommodated by either elastic deformation and/or the expulsion of unfrozen water. If an aggregate is less than critically saturated, or can expel water to its surroundings, internal hydraulic pressures result as unfrozen water is forcibly displaced by ice formation. The hydraulic pressures are greater when this flow is through smaller sized pores and along longer flow paths (Winslow 1994). The tensile capacity of an aggregate can be exceeded when either the expansion due to ice formation cannot be accommodated by elastic deformation or the hydraulic pressures become excessive. In other cases, the expulsion of water from sound aggregate particles during a freezing event can result in damage to the surrounding paste phase.

It is fairly well established that the aggregate pore system is the most important factor contributing to its freeze-thaw durability (Schwartz 1987; Winslow 1994; Pigeon and Plateau 1995). Two related elements of the pore system that are of particular interest are the porosity and pore size distribution. Aggregate porosity is the ratio of the surface accessible pore volume to the total aggregate volume. Porosity can be easily determined, providing a measure of the total volume of water that can be contained within a fully saturated aggregate. Pore size is a measure of the physical dimensions of the various elements of the accessible pore volume. It is a much more difficult characteristic to determine, but provides an important indication of an aggregate's potential to become critically saturated. It also provides a measure of its permeability or resistance to fluid flow during a freezing event.

Perhaps the single most significant pore characteristic is the volume of the total porosity contained within a range of pore sizes. This parameter has been empirically related to aggregate freeze-thaw durability (Kaneuji 1980). The exact limit of detrimental pore sizes is not definitively established, but has been reported to be in the range of under $0.10\text{ }\mu\text{m}$ up to $5\text{ }\mu\text{m}$ (Kaneuji et al. 1980; Marks and Dubberke 1982; Shakoor and Scholer 1982). Elements of the pore system smaller than the lower limit are not considered detrimental for one of two reasons. Either their size is such that the water in them will not freeze at normal winter temperatures or the contained volume of water is so small that it does not contribute significantly to stresses in either the aggregate itself or the surrounding paste. Pore systems with sizes greater than the upper limit are not considered detrimental to the aggregate because either they do not become critically saturated or their permeability is so high that they do not generate excessive hydraulic pressures. Aggregates with a preponderance of pore sizes within the detrimental size range are believed to have a higher potential to become critically saturated and/or develop excessive hydraulic pressures due to low permeability.

Because internal hydraulic pressures are directly proportional to the length of the flow path, reducing the size of the aggregate can reduce the magnitude of these internal pressures. Reducing the aggregate size can also result in a smaller volume of expelled water per unit surface area. Experimental studies have confirmed an increased freeze-thaw durability of susceptible aggregates with a reduction in maximum particle size (Stark and Klieger 1974). The maximum particle size required to ensure a freeze-thaw durable aggregate varies greatly because it depends on the characteristics of the aggregate pore system, including the porosity and the pore size distribution (which will affect the degree of saturation and permeability), along with the mechanical properties of the aggregate. Additionally, the properties of the surrounding paste system influences the value of this maximum size. While reducing the maximum particle size may improve the freeze-thaw performance of some aggregates, it also typically results in an increased cement paste/mortar volume. In addition to being more expensive, a higher paste volume can increase drying shrinkage and result in reduced aggregate interlock at joints and across structural cracks. It also will increase the total alkalinity per unit volume of concrete and thus may contribute to alkali-aggregate reactivity (Leming 1996). The fine aggregate fraction is likely unassociated with aggregate-related freeze-thaw deterioration due to their size.

A.2 Deterioration Due to Chemical Attack Mechanisms

Alkali–Silica Reactivity

Alkali–silica reactivity (ASR) is a distress caused by undesirable chemical reactions between alkalis in the cement paste (Na_2O and K_2O) and the reactive siliceous components of susceptible aggregates. The product of the reaction is expansive in the presence of moisture, destroying the integrity of the weakened aggregate particle and the surrounding cement paste (Mindess and Young 1981). The reaction product is often referred to as a gel, which is a misidentification in some instances. The term gel implies an amorphous structure when in fact, often, the product is crystalline. ACI (1998) provides an in-depth description of this chemical attack mechanism.

With time, the alkalis in the paste pore solution may react with certain forms of reactive silica contained in the aggregate forming the alkali-silica reaction product. The most reactive forms of aggregate are strained quartz, amorphous silica, cryptocrystalline quartz, chalcedony, and chert. If water is available, the reaction product will draw it out of the surrounding cement paste, causing swelling, exerting potentially damaging tensile stresses from 4,100 to 11,000 kPa within the cement matrix (Farny and Kosmatka 1997). These stresses may also be large enough to cause bond and shear failures between concrete and reinforcement (CCAA 1996).

The penetration of hydroxyl ions into the siliceous aggregates breaks the silicon-oxygen linkages within the silica tetrahedra. The resultant negatively charged silicon hydroxide ions are electrically balanced through interaction with the sodium and potassium ions in the pore solution, forming the new reaction product (Harrison et al. 1987). Swelling occurs as the forces of attraction between the polar water molecules and the alkali–silicate ions results in chemically bound water in the solid reaction product.

Factors that affect the rate at which ASR develops include the amount and type of siliceous materials present, the concentration of alkalis in the pore water solution, and the amount of moisture accessible to the gel product. At higher temperatures, gel expansion is greater but stabilizes earlier (CCAA 1996). Certain rock types containing silica are more reactive than others. In addition to the minerals named previously, the following rock types are known to be potentially reactive: rhyolite, dacite, latite, andesite, tuffs, shale, slate, sandstone, siltstone, quartzite, granites, grano-diorites, and granite gneisses. This list is by no means complete, but it shows that a large number of rocks may be reactive and deleterious to concrete (Farny and Kosmatka 1997). In general, aggregates containing crystalline silica are stable and those with amorphous or very fine grained silica are reactive. Also, aggregate with large surface areas or many lattice defects are more susceptible to ASR. Thus, the potential reactivity of an aggregate is a function of both its degree of crystallization and the amount of energy stored in the crystal structure (Leming 1996).

The concentration of alkali in the pore solution is the second factor affecting the rate of ASR development. This factor is affected by two main sources: the alkali content of the cement and alkali addition from other sources, both internal and external. The alkali content of cement is normally expressed as Na_2O equivalent alkalis (Na_2O percent + 0.658 percent K_2O). Low alkali cement is typically considered to have an Na_2O equivalent of 0.6 percent or less. But it is noted that the total alkalis from the cement in a given concrete is the product of the Na_2O equivalent and the cement content. Thus, a mixture made with low alkali cement may in fact have high total alkali content if a large amount of cement is used. For this reason, European standards specify a limit of 3 kg/m³ on the total alkali content per unit volume of concrete (Leming 1996). Internal sources of alkalis other than the cement include mineral or chemical admixtures, aggregates, and even the mixing water. External sources of alkalis include deicing salts, seawater, groundwater, and water from industrial processes that permeate the concrete (Farny and Kosmatka 1997).

It is noted that a highly alkaline pore solution is not necessary for ASR to develop. Aggregate composed of highly reactive silica may suffer ASR even if the pore solution has a relatively low concentration of alkalis. Alternately, if the alkali concentration of the pore solution is high enough, it can break the strong

silica bonds present in less reactive aggregates resulting in deleterious ASR (Farny and Kosmatka 1997). ASR has also been known to develop in areas of localized high alkalinity even if the alkalinity of the rest of the pore solution is low.

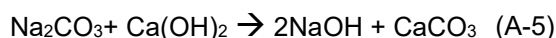
The third factor affecting the expansion of the ASR reaction product is the amount of accessible moisture. It has been found that an internal relative humidity of at least 75 to 80 percent is needed for deleterious ASR to occur (Farny and Kosmatka 1997; CCAA 1996). This internal relative humidity is common in pavement concrete. It is still unclear whether continuous saturation or cycles of wetting and drying cause more ASR-associated damage (Farny and Kosmatka 1997; CCAA 1996).

Alkali–Carbonate Reactivity

As discussed in the previous section on ASR, the pore solution of concrete is highly alkaline, primarily a result of the sodium and potassium ions released during cement hydration. Excess alkalis are also derived from admixtures, mixing water, fly ash, GGBFS, silica fume, and external sources (CCAA 1996). The exact mechanism of ACR is still debated, but it is believed that these alkalis react with the carbonate aggregates in a process called dedolomitization, a breaking down of the dolomite to form brucite, calcium carbonate, alkali carbonate, and alkali hydroxide (Farny and Kosmatka 1997; ACI 1998). The type of aggregates susceptible to this type of reaction are typically dolomitic limestone that consist of a fine-grained matrix of calcite and clay in which larger crystals (20-80 μm) of euhedral dolomite rhombohedra are suspended (Ozol 1994). Dedolomitization can be represented by the following equation in which M represents potassium, sodium, lithium or other alkali element (Ozol 1994):



The alkali carbonate produced in this reaction may then react with $\text{Ca}(\text{OH})_2$ produced through normal cement hydration to regenerate the alkali hydroxide, an example of which is illustrated in the following equation (Ozol 1994):



Although the starting and end products of this reaction are well documented, the expansion portion of the reaction is still being debated. There are two main theories on how the expansion occurs in concrete. The first theory suggests that the expansion is caused by migration of alkali ions and water molecules into the restricted space of the fine-grained matrix surrounding the dolomite rhomb. Typically, the alkali reactivity of carbonate rocks is not dependent on the clay mineral composition (Farny and Kosmatka 1997). This theory argues that expansion is caused when clay, exposed by dedolomitization, attracts and absorbs water.

The second theory is that the growth and rearrangement of the de-dolomitization products, especially brucite, exerts pressure as it crystallizes. This pressure from the crystallizing products causes expansion (Farny and Kosmatka 1997; Gillott 1995). Leger et al. (1995) and Nixon and Page (1987) suggest that the first theory is correct and that an expansive reaction occurs with clay minerals. On the other hand, Mingshu et al. (1994) added credence to the second theory by showing that de-dolomitization itself causes expansion. Ozol (1994) suggests that both theories might be correct, stating that ACR expansion is a result of a combination of both mechanisms. The literature suggests that more research is required before the absolute mechanism of deterioration is fully understood.

Regardless of the mechanism, it is known that for the ACR reaction to occur, three criteria must be met. First, the content of reactive aggregate must be in excess of a critical value. Second, there must be

sufficient alkalis present in the pore solution of the concrete to initiate a reaction. Lastly, sufficient moisture must be available so that the reaction may proceed (Leger et al. 1995). As with ASR, ACR requires a source of moisture in order to have deleterious consequences. Thus ACR is not common if the concrete has an internal relative humidity less than 75 percent (CCAA 1996).

Swamy (1994) identified three aggregates with which carbonate reactions commonly occur. These aggregates are calcitic limestones, dolomitic limestones, and fine-grained dolomitic limestone aggregates containing interstitial calcite and clay. Farny and Kosmatka (1997) outline aggregates that have potential for ACR in the following manner:

- Aggregates with a clay content, or insoluble residue, in the range of 5 to 25 percent.
- Aggregates with a calcite to dolomite ratio of 1:1.
- Aggregates with increasing dolomite volume up to a point at which interlocking texture becomes a restraining factor.
- Aggregates with discrete dolomite crystals (rhombs) suspended in a clay matrix.

In ACI (1998), the concept of early expanders and late expanders is discussed. The difference between the late and early expanders is reflected in the bulk composition and internal textural restraint. Late expanders typically have a higher acid insoluble residue (21 to 49 percent) and a higher percent of dolomite present (75 to > 90 percent) of total carbonate (ACI 1999). The degree to which reactive carbonate rocks will cause expansion in concrete is related to the restraint imposed by the concrete, the volume of dolomite in the rock, and the internal textural restraint of the carbonate rock (Ozol 1994). Ozol (1994) also reports that the expansion of concrete containing alkali reactive carbonate rocks is promoted by increasing coarse aggregate size, moisture availability, higher temperature, high alkali content of concrete, high pH of the liquid phase in cement pores, high proportion of reactive stone in coarse aggregate, and lower concrete strength.

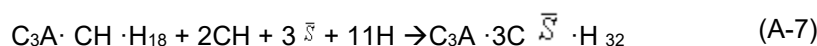
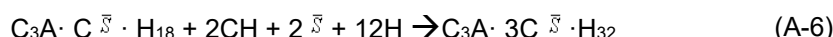
Sulfate Attack

As with most other MRD mechanisms, the basic mechanism of sulfate attack has been thoroughly investigated but not fully understood (DePuy 1994). The first step in the process is that sulfate ion become available within the concrete pore solution, either through diffusion of sulfate ions into the concrete from the outside or through internal sources. The permeability of the concrete and the diffusion coefficient of the sulfate ions control this process. Microcracking, which may result from the sulfate attack or other causes, increases the permeability of the concrete, leading to acceleration of the damage (Mindess and Young 1981). However, Hughes (1985) found that susceptibility to sulfate attack was not simply a function of water permeability. He proposed that resistance to sulfate attack is related specifically to the ability of the sulfate ion to diffuse through the pore structure. This is not only related to permeability, but also to the ability of the sulfate ions to then gain access to the susceptible minerals. This in turn is related to the entry size of the pores associated with those minerals. Gowripalan et al. (1993) cite several studies that conclude that the flow of a fluid through concrete is related to the larger capillary pores and not the total porosity; Scrivener (1996) found that the interfacial transition zones around aggregate particles are an important factor in the permeability of the concrete. Thus, the volume

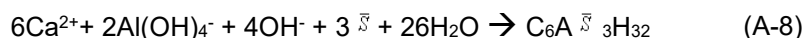
of larger pores and the characteristics of the interfacial zone are correlated with the concrete's resistance to sulfate attack.

Once sulfate ions have diffused into the pore system, the conversion of sulfoaluminates to expansive ettringite, also known as calcium trisulfoaluminate ($3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 32\text{H}_2\text{O}$) can occur. This is the cause of most of the disruption of the concrete matrix due to sulfate attack (ACI 1992a). The exact mechanism of this disruption is still controversial. Mehta and Monteiro (1993) state that there is general agreement that expansion resulting from sulfate attack is related to ettringite formation that occurs when sulfate ions attack the calcium hydroxide and alumina-bearing phases of a hydrated portland cement paste. Therefore, many researchers have correlated the sulfate resistance of cement to its C_3A content because it controls the amount of sulfoaluminate hydration products that will form in the paste. Mindess and Young (1981) agree with this, stating that the susceptibility of concrete to sulfate attack is related to how much ettringite can form, which relates back to the amount of C_3A in the cement.

Mehta and Monteiro (1993) state that portland cements with more than 5 percent potential C_3A will contain most of the alumina as monosulfate hydrate ($\text{C}_3\text{A} \cdot \text{CaSO}_4 \cdot \text{H}_{18}$). If the C_3A content is more than 8 percent, the hydration products will also contain $\text{C}_3\text{A} \cdot \text{CH} \cdot \text{H}_{18}$. In the presence of calcium hydroxide and sulfate, both of the alumina-containing hydrates are converted to the high sulfate form of ettringite, according to the following reactions:



Bickley et al. (1994) present the ettringite formation reaction in the following manner:



They state that the required $\text{Al}(\text{OH})_4^-$ can be supplied by C_3A or C_4AF in the un-reacted clinker, but is more likely from the monosulfoaluminate ($3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{CaSO}_4 \cdot 18\text{H}_2\text{O}$). Wolter (1997) states that ettringite can also form from gypsum reacting with the C_3A . Calcium monosulfoaluminate ($3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{CaSO}_4 \cdot 12\text{H}_2\text{O}$) forms if the concentration of sulfate is below the point at which ettringite is stable (DePuy 1994; Chatterji 1969). In a higher sulfate solution, the calcium monosulfoaluminate is converted to ettringite (DePuy 1994). In both of these forms, part or all of the Al^{3+} ions can be replaced by Fe^{3+} ions (Chatterji 1969).

Although there is general agreement that sulfate-related expansions in concrete are associated with ettringite, how the ettringite causes expansion is still a subject of controversy. Exertion of pressure by growing ettringite is one theory, supported by Wolter (1997) and Lafuma (in Chatterji 1969). Lafuma's theory is that ettringite formation can take place through two different mechanisms. If the liquid phase containing sulfate ions is saturated with CH, then ettringite is formed in a solid-state reaction between C_3A

and the SO_4^{2-} and Ca^{2+} ions in solution. This reaction causes a volume increase of about 815 percent on the basis of C_3A volume. If the liquid phase is unsaturated with respect to CH, the reaction occurs in the liquid phase and ettringite formation occurs only in the existing pores in the structure without causing expansion (Chatterji 1969).

Swelling due to adsorption of water in an alkaline environment by poorly crystalline ettringite is another theory supported by Mehta (1991) and Wolter (1997). Mehta (1991) cites a study that provides evidence of swelling of microcrystalline ettringite. Additionally, Wolter (1997) speculates that ettringite needs a high pH environment in order to crystallize. This is based on observations that in concrete with extensive ettringite, there is no ettringite where carbonation has occurred. Wolter (1997) speculates that the ettringite may dissolve and move to an area of higher pH and recrystallize as secondary ettringite in available void space.

The formation of thaumasite ($\text{CaCO}_3 \cdot \text{CaSO}_4 \cdot \text{CaSiO}_3 \cdot 15\text{H}_2\text{O}$), a mineral that is structurally similar to ettringite, may also contribute to sulfate attack. It has been speculated that thaumasite is not often identified in concrete because it is often misidentified as ettringite due to their similarity in structure and appearance (Rodgers et al. 1997). Berra and Baronio (1987) describe it as a complex sulfate salt, a heavily hydrated triple compound of calcium metasilicate, calcium sulfate, and calcium carbonate. It is similar to ettringite, except that silica is substituted for alumina (Crammond 1985; Berra and Baronio 1987). DePuy (1994) states that it may form as a conversion product of ettringite with carbonation and silicon substitution, or directly when there is a supply of alumina, calcium silicates or free silica gel, sulfate, and carbonate.

Thaumasite forms preferentially at low temperatures because of the high solubility of calcium salts at low temperatures (DePuy 1994). It may also form due to the higher solubility of CO_2 at cold temperatures, which favors a carbonation reaction necessary for thaumasite formation (Bickley et al. 1994). At 5°C , thaumasite is a hundred times less soluble than it is at 20°C , which may partially explain some cold temperature deterioration (Crammond and Halliwell 1995). For this reason, it is thought to be more prevalent in colder climates such as the northern U.S. and Canada (Rodgers et al. 1997).

Unlike ettringite formation, the formation of thaumasite is dependent on the more abundant supply of CSH and not on the C_3A content. This means that the deterioration is not affected nor limited by C_3A content, and can proceed until there is no binding agent left, completely destroying the concrete. Thaumasite formation has been found to occur more readily with dolomite aggregates, attributed to the dedolomitization of the aggregate that provides additional CaCO_3 (Crammond and Halliwell 1995). Crammond and Halliwell (1995) suggest that all types of PCCs containing finely divided carbonate rock dust as a source of carbonate ions may be potentially vulnerable to thaumasite attack, based on very limited evidence.

In addition to formation of ettringite or thaumasite, sulfate attack also occurs due to the formation of gypsum. This mechanism is commonly referred to as gypsum corrosion. Gypsum corrosion only contributes directly to the deterioration of the concrete at sulfate concentrations above 1,000 ppm and is only of secondary importance until sulfate levels exceed 4,000 ppm (Mindess and Young 1981). Gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) formation takes several forms, depending on the form of the attacking sulfate. With relatively small concentrations of sulfate ions, the nature of the sulfate ion does not matter. But as ion concentration increases, the nature of the ion has an influence. At lower concentrations ($\text{SO}_4 < 1000 \text{ mg/L}$), sodium sulfate reacts to form ettringite, and at higher concentrations, it produces gypsum (DePuy 1994). For sodium sulfate gypsum corrosion, Mehta and Monteiro (1993) gives the following reaction:



In this case, the formation of sodium hydroxide as a byproduct of the reaction maintains the high alkalinity of the system, protecting the stability of the CSH phase.

External Sulfate Attack

The most common source of external sulfates is from soils or groundwater. Impurities in deicing chemicals can also provide a ready source of external sulfates. This has been labeled deicer distress and has been gaining attention in some northern States. Sulfates are a common impurity in many naturally occurring deicing salts. Standard NaCl deicing salts are required to be as little as 92 percent pure, allowing for up to 8 percent impurities. These impurities are often gypsum ($\text{CaSO}_4 \cdot \text{H}_2\text{O}$) and anhydrite (CaSO_4) (Wolter 1997). One research project found that natural rock salt may contain as much as 4.0 percent calcium sulfate, which increases in solubility in a chloride solution (Pitt et al. 1987). The amount and increased solubility of the calcium sulfate was found to be sufficient to cause deterioration to concrete mortar.

According to Wolter (1997), exterior concrete exposed to moisture and a natural deicing agent with gypsum or anhydrite impurities under freeze-thaw conditions can be potentially affected by deicer distress as the gypsum impurity in the deicer reacts with the C_3A to form ettringite. As Munoz and Chou (1996) state, the gypsum reacts with the hydrated calcium sulfoaluminates and calcium aluminates as described previously. Deicer distress is different from a standard sulfate attack in that the gypsum is introduced directly, without the reaction between the sulfate and calcium hydroxide (Wolter 1997). Under these conditions, it is speculated that the presence of gypsum leads directly to ettringite formation as described above.

In addition to deleterious expansion, the ettringite may also go into solution, recrystallizing within the pore space and air void system of the concrete. The process of void infilling is incremental, as air voids would only become partially filled with water as it moves from the paste to relieve hydraulic/osmotic pressures developed during freezing. As the water enters the air void, it forms ice crystals on the pore wall. When the crystals melt, secondary products are deposited on the pore walls (Niemann and Lehtonen 1997). Over time, incremental deposition of ettringite and other secondary deposits on the walls of air voids may completely fill the interstitial pore space and air-void system, leaving the concrete vulnerable to freeze-thaw deterioration.

The incremental filling of the voids and resulting vulnerability of air-entrained concrete to freeze-thaw damage has been verified through an extensive study conducted in Sweden (Niemann and Lehtonen 1997). Over 1,500 thin sections were petrographically analyzed. Scanning electron microscopy and energy dispersive x-ray microanalyses were used to study the deposits as well as judge the extent of void infilling. This study found that void infilling was progressive, with increasing void volume being filled over time and smaller voids were filled first. This study also demonstrated a decrease in freeze-thaw durability with an increase in volume of voids filled. The density of the material filling the voids approached that of the aggregate, and the study concludes that these voids were no longer able to protect the concrete against freeze-thaw damage. Similar results were found in a German study (Bollmann and Stark 1996).

Dubberke (1993) gives a similar, but slightly different, explanation for the deterioration of Highway 520 in Iowa. He theorizes that the pores became filled with "amorphous", non-crystalline ettringite. He suggests that expansion could occur from the generation of the amorphous ettringite paste in small pores (<50 microns), first in high moisture areas such as the bottom of the slab and near joints. This causes differential expansive forces between the top and bottom of the slab, leading to cracking. The cracking occurs in the weakest areas, such as the vibrator trails that lack coarse aggregates and have a relatively high w/c ratio. However, it has been noted by some that the expansive pressures generated by secondary deposition of ettringite in available void space is insufficient to cause damage to hardened concrete (Scrivener 1996; Johansen et al. 1993).

It must be noted that finding ettringite in microcracks in the paste is in itself not sufficient evidence to link the distress mechanism to external sulfate attack. Ettringite is a common component of hydrated cement paste and the question arises as to whether the observed cracking was caused by the ettringite formation

or if secondary ettringite merely formed in the available space (DePuy 1994). This makes diagnosis of external sulfate attack difficult in some cases.

Internal Sulfate Attack

The term internal sulfate attack (ISA) is used to describe the deterioration of concrete resulting from sulfate attack where the source of the sulfate ions is from within the concrete itself. DePuy (1994) describes sulfates that may attack concrete. Potassium sulfate and sodium sulfate strongly react with PCC. Sulfates having very low solubility in water, such as barium and lead sulfate, which are common in some aggregates, are relatively harmless. Calcium sulfate is less aggressive than sodium sulfate, and magnesium sulfate is the most aggressive (Mielenz 1964). Although all of the reactions previously described in the discussion of external sulfate attack apply, and many sulfate phases can form within concrete, it is the formation of ettringite that is clearly the most important issue related to internal sulfate attack.

Ettringite is a normal meta-stable phase produced in the cement hydration process. It forms from the reaction of C_3A or monosulfoaluminate with sulfate ions. The formation of ettringite by alteration of an existing solid phase is an expansive formation and, if the crystals are allowed to form while confined in the cement paste, a process of cracking will initiate within the paste.

To help establish a standard nomenclature on this issue, the American Concrete Institute's Committee 116 (Names and Definitions) is working on definitions for the various forms of ettringite. Preliminary definitions are presented in table A-1 (Erlin 1996a).

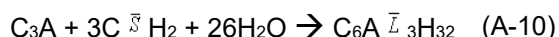
Delayed ettringite formation (DEF) and excess sulfate (EXS) are two types of internal sulfate mechanisms identified by Thaulow et al. (1996a). Secondary ettringite formation (SEF) is a third potential mechanism of internal sulfate attack. These have been increasingly identified as the cause of many cases of distress in PCC. DEF is most often associated with steam curing. At elevated temperatures (current research suggests a minimum temperature of 65 C to 80C [Scrivener 1996; Thaulow et al. 1996a; Klemm and Miller 1997]), primary ettringite will not properly form. After the concrete has cured and temperatures are reduced to ambient conditions, sulfates and aluminate phases in the paste may then react to form expansive ettringite, disrupting the concrete matrix. Because this phenomenon is most closely associated with steam curing, it is still speculative whether cast-in-place pavements can experience the temperatures necessary to produce DEF. EXS, on the other hand, appears to be a more likely mechanism for ISA. Possible sources of internal sulfate include either slowly soluble sulfates or sulfur compounds in the clinker or fly ash that only become available during continued long-term hydration. Another internal source of sulfates might be from aggregates (Johansen and Thaulow 1999). In these cases, the internal sulfate attack is not from DEF, but instead from excess sulfates in the concrete mixture, which result in paste expansion along similar lines as external sulfate attack.

Table A-1 Preliminary definitions of forms of ettringite being considered by ACI Committee 116—names and definitions (Erlin 1996a).

Form	Definition
Ettringite	A high sulfate calcium sulfoaluminate mineral ($3\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot 3\text{CaSO}_4 \cdot 32\text{H}_2\text{O}$).
Primary Ettringite	Ettringite formed by reaction of sulfate and aluminate ions during early hydration of hydraulic cement either as a normal process for portland cement or as the expansive process for expansive cement.
Secondary Ettringite (SEF)	Ettringite formed by precipitation from solution of primary ettringite.
Delayed Ettringite (DEF)	Ettringite formed by reaction of sulfate and aluminate ions in concrete, mortar, or grout that has hardened and developed its intended strength. If the sulfate is from an external source, the process is "sulfate attack;" if the sulfate is from within the concrete, the process is "delayed ettringite formation."

Currently, there is no consensus among researchers about possible SEF distress mechanisms. Also, there is considerable debate about the internal sources of sulfate, with portland cement, aggregates, and fly ash all identified as possible sources. Based on the given definitions, it is evident that the composition of the cement may play a role in internal sulfate attack, but compositional variations are difficult to isolate due to the numerous producers, variations in production methods, and the range of raw materials and fuel sources used to produce portland cement. Other sources of sulfate, such as fly ash and aggregates, are being limited in many cases by strict specifications in an effort to control internal sulfate attack.

From the literature, it is almost impossible to separate DEF from EXS, particularly in nonsteam-cured structures such as pavements. As a result, these two will be treated together unless the mechanism at work is clearly identified. The basic mechanism of deterioration in DEF/EXS is the bulk expansion of the paste caused by reactions between solid phases within the cement paste and internally derived sulfate ions. In the DEF/EXS reaction, at least one of the reactants (i.e., usually aluminate or ferrite cement clinker phases or monosulfoaluminate) begins the reaction as a solid within the hardened cement paste and undergoes a solid-solid phase transformation with a significant increase in volume. This results in crack initiation. Mindess and Young (1981) give the following reaction for the conversion of tricalcium aluminate (C_3A) and gypsum to ettringite:



Note that this reaction is a normal occurrence in the early hydration of portland cement. The crystallization pressure of ettringite forming from un-hydrated C_3A is estimated as high as 240 mPa (35,000lb/in²) and is sufficiently large enough to cause cracking in concrete (Mindess and Young, 1981).

Portland cement clearly contains phases, such as tricalcium aluminate, that serve as the principal source of aluminate for ettringite formation. More importantly, although aggregates and external sources of sulfate can contribute significantly to the total sulfate in the concrete, portland cement is the primary source of the sulfate required to form ettringite (Erlin 1996b). For the most part, the constituents of portland cement have not changed over the years and cement producers argue that with modern quality control procedures, a more uniform product is now being produced than in the past. However, the sulfate content of portland cement has increased over the years. A study conducted by the PCA indicates that between the 1950s and 1994, the average sulfate contents in portland cement have increased almost 58% from 1.9% SO_3 to 3.0 % SO_3 for Type I cements (PCA 1996). Similar increases were noted for Type II cements.

At least part of this increase in sulfates is due to higher amounts of gypsum being added to address the more rapid reactivity of finely ground modern cements. But it has been hypothesized that some of this increase in sulfate content is a result of burning waste fuels. Such sulfate would be included in solution in the silicate phases of the cement clinker (Mielenz et al. 1995; Hime 1996, Wolter 1997). It is speculated that this sulfate is slowly soluble and would not contribute to the total sulfate available to form primary ettringite until coming out of solution at the latest stages of hydration. This hypothesis has gained acceptance due in part to a well-publicized case of distress in 400,000 steam cured cement railroad ties in the northeastern part of the United States (Mielenz et al. 1995). In this case, a team of consulting engineers determined that the majority of the ties failed as a result of DEF/EXS and that the sulfate source for ettringite formation was anhydrite phases and sulfate in solution in the cement clinker silicate phases. It is noted that this hypothesis is contested, and that the observed DEF had less to do with clinker sulfates than it did with curing practices, particularly high curing temperatures (Scrivener 1996).

Another point of contention is the classification method used to describe fly ash. Current methods (ASTM C 618) fail to adequately characterize fly ash that may be deleterious. Prescriptive tests that determine the soluble sulfate content and the soluble alumina content are needed (Dewey et al. 1996). The application of the former test is obvious, as knowing the total sulfate content in a mix is essential to predicting the formation of ettringite. In the latter case, the alumina component of the fly ash must be easily soluble to prevent alumina rich inclusions from forming in the CSH. If such inclusions form, they may be susceptible to sulfate attack and expansion (Gress 1997).

A key variable controlling DEF is the conditions under which the concrete is cured. In particular, the temperature of the concrete during curing appears to be an important factor influencing the development of delayed ettringite within that concrete. Concrete distress as a result of DEF is most common when the concrete is cured at elevated temperatures (i.e., steam curing) (Day 1992). As a matter of definition, Thaulow et al. (1996a) state that elevated temperatures are absolutely necessary for DEF to occur. The initial mechanism suggested for the occurrence of DEF at elevated temperatures is the thermal decomposition of primary ettringite formed during the early stages of hydration. After the concrete has cooled, delayed ettringite forms from the decomposition reaction products and causes the cement paste to expand, creating gaps around coarse aggregate particles. It is within these cracks that secondary ettringite may form, filling this newly created volume (Skalny 1996).

Although the high temperatures needed for DEF are normally associated with steam curing, some researchers have reported DEF at concrete curing temperatures obtainable during cast-in-place construction. The internal temperatures in pavements cured under high ambient temperatures may approach the critical temperature for DEF, which current research suggests is a minimum temperature of 65°C to 80°C, with many citing 70°C (Scrivener 1996; Thaulow et al. 1996a; Klemm and Miller 1997). It has been proposed that excess heat from the exothermic hydration process can increase the chances of DEF occurring (Hime 1996). Scrivener (1996) indicates that in the presence of excess sulfate, elevated temperatures (60 to 100°C) will increase the chance of DEF and that the ettringite that forms is microstructurally different than ettringite formed as a result of high sulfate content only. This research is consistent with field evidence that DEF is most often occurs in steam-cured concrete.

However, another researcher has shown that DEF is seen in nonsteam-cured concrete and that its microstructure is similar to ettringite formed in concrete cured at lower temperatures (Diamond 1996). Gress (1997) indicates that some cements are known to form delayed ettringite at temperatures as low as 45°C which is clearly attainable in slab-on-grade construction under slightly adverse weather conditions, especially if a high cement content mix is being used. These instances of internal sulfate attack may be more closely related to EXS and further research is required.

Internal sulfate attack by SEF is distinguished from that previously described under external sulfate attack through consideration of the source of the sulfate ions. If the source is from a concrete constituent or dissolved primary or delayed ettringite, it is considered to be from an internal source. If the sulfate source is from an external source, such as from groundwater or deicer impurities, the SEF would be considered as an external sulfate attack.

In SEF, the ions reacting are in the pore water and precipitate out of solution as ettringite into available spaces such as voids and cracks. If during petrographic examination ettringite is observed in cracks, there is a question as to whether it caused the cracking or merely formed in the available space created through the action of another deterioration mechanism (DePuy 1994). Given that SEF is common in older, hardened concrete, it is at times difficult to attribute it to the observed distress characteristic of paste expansion. In the case where a crack or gap is filled with ettringite, it is often unclear whether the crack or gap came first or did expansion of the ettringite cause the defect. For example, in a recent publication, Erlin (1996b) suggests that the cement paste expansion can result from any number of phenomena including freeze-thaw cycling, hydration of free lime and magnesia, and ASR. Once expansion and cracking occur, secondary ettringite can form and expand into the newly created cracks and voids.

More quantitatively, another researcher has shown using x-ray diffraction, that no ettringite was present in cement mortars immediately after steam curing. However, after the concrete had hardened and a crack structure had formed, primary and secondary ettringite formed to fill the available space (Scrivener 1996). The same researcher also presents reasons why secondary ettringite forms in cracks, rather than creating cracks. The principal reason given is that the level of sulfate saturation in the pore water system of normal concrete does not meet that required to produce a crystallization pressure large enough to cause cracking (Scrivener 1996). The concept of crystallization pressure is also discussed in detail by Johansen et al. (1993) who agree that the necessary conditions for crystallization pressures sufficient to cause cracking in hardened concrete do not exist for the case of SEF.

Ettringite loses a considerable amount of water upon drying (Mindess and Young 1981). Given this fact, swelling of poorly crystalline secondary ettringite due to absorption of water in an alkaline environment is another theory of expansion supported by Mehta (1991) and Wolter (1997). Mehta (1991) cites a study that provided experimental evidence of swelling of microcrystalline ettringite. As previously discussed, Wolter (1997) and Niemann and Lehtonen (1997) suggest that SEF can completely fill the air-void system to the point where it is no longer available to protect the paste from freeze-thaw damage. Thus deterioration occurs due to freeze-thaw action resulting from SEF.

Corrosion of Embedded Steel

Corrosion of embedded steel manifests as scaling, cracking, and associated deterioration of the concrete in areas above or surrounding areas affected by active corrosion. A film of tightly adhering corrosion products normally helps protect reinforcing steel within a concrete structure from continued corrosion (Perenchio 1994). Depassivation of the steel requires that the calcium hydroxide crystals typically present at the steel-cement paste interface are first decomposed and removed. This is followed by the destruction of the passive iron oxide film (Mehta 1991). This film is thermodynamically stable at pH levels greater than 11 (Mehta 1991). Once this film is breached, however, either through a reduction in concrete alkalinity or due to the presence of chloride ions, the deleterious corrosion process can occur. Typically, corrosion due to chloride ingress proceeds at a rate much faster than carbonation due to reduction in alkalinity (e.g. carbonation of the surrounding paste).

The mechanisms involved in the corrosion of steel reinforcement can be identified and explained using basic chemical and physical processes. When any metal corrodes, in essence, it is returning to its natural state as an oxide or hydroxide. Very few metals are found naturally in their metallic state because it is their highest energy state. Since all materials seek to exist at their lowest energy level, the metals will naturally oxidize if possible. This is what drives the corrosion of steel reinforcement (Perenchio 1994). ACI 222R (1989) provides an in-depth description of this corrosion mechanism.

Three components, the anode, the cathode, and the electrolyte, are necessary for a corrosive electrolytic cell to exist. The anode is the site where the metal corrosion occurs through the loss of electrons and oxidization. The anodic process cannot occur until the passive iron oxide (Fe_2O_3) film is either removed due to lowering of the concrete alkalinity or made permeable in the presence of chloride ions (Mehta and

Monteiro 1993). The cathode is the site where the excess electrons from the anode are consumed. It is said that the metal at this site is reduced. The cathodic process requires both oxygen and water to be present at the steel surface (Mehta 1994). Finally, an electrolyte is needed to couple the anode to the cathode. The electrolyte allows the exchange of electrons, and is defined as the medium through which the current that drives the anodic and cathodic reactions flows (Fraczek 1987). The corrosion process is illustrated in figure A-1.

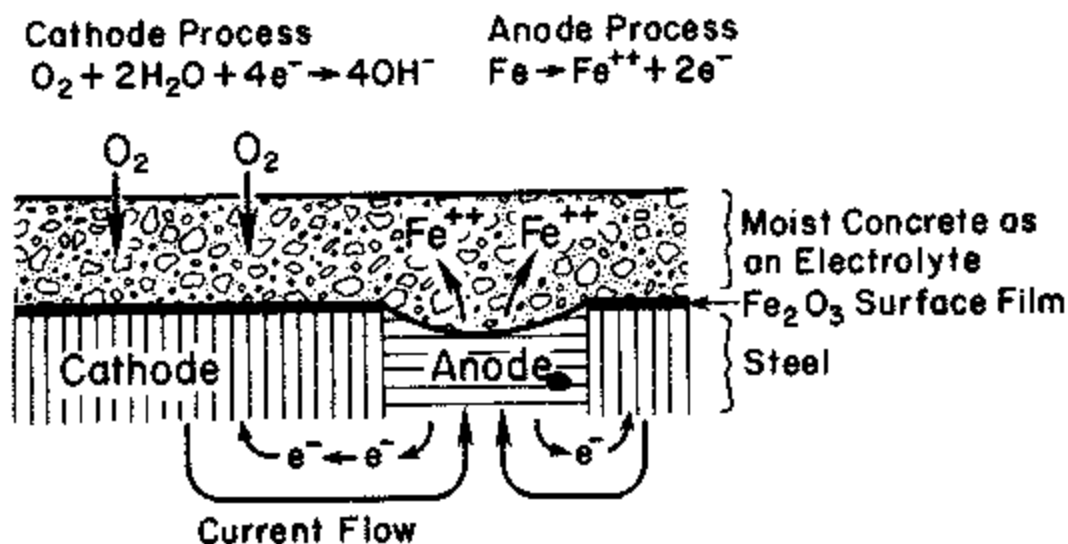
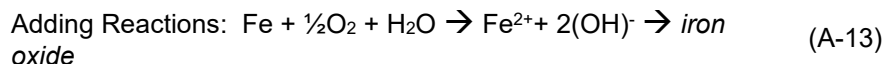
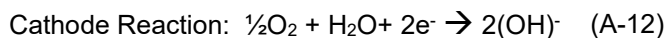
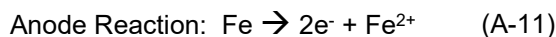


Figure A-1. The electrochemical process of steel corrosion (Mehta and Monteiro 1993).

The following equations describe the electrochemical process involved in steel corrosion (Mehta 1991):



Corrosion of reinforcing steel occurs in concrete as an electrolytic cell is formed between the reinforcing steel and the surrounding concrete. The concrete matrix acts as the electrolyte of the cell by allowing the transfer of electrons between the anode and the cathode. The anode and the cathode of the electrolytic cell may be formed within the structure either on the same dowel, tie, or reinforcing bar or on two separate bars in the concrete. To accomplish this an upper reinforcing bar or dowel will act as the anode, giving up some of its electrons so it can oxidize. A lower bar will act as the cathode having access to oxygen from the bottom of the slab. The reaction product formed from the corrosion of the upper reinforcing steel is iron oxide, commonly known as rust. Rust is responsible for most of the damage

caused by corrosion, as its formation is an expansive reaction. Because the reinforcement is tightly embedded within the concrete, the expansion results in cracking and spalling.

Free chloride ions are a major contributing factor in the corrosion of reinforcing steel. Free chloride comes from many sources, including deicing salts, aggregate, cement, mixing water, and chemical admixtures (Perenchio 1994). In the presence of moisture and oxygen, chloride ion concentrations in excess of a threshold level can accelerate corrosion (ACI 1992a). The exact concentration of chloride ions that constitutes this threshold level is debated, but the general consensus is that some chloride ions can be tolerated (ACI 1989). A number of studies cited in ACI 222R (1989) found that corrosion of embedded steel was initiated if the acid-soluble chloride concentration exceeded 0.20 percent. Mehta and Monteiro (1993) suggest the threshold value is in the range of 0.6 to 0.9 kg of chloride ions (Cl^-) per cubic meter of concrete. They also note that the hygroscopic nature of chlorides will increase the saturation level of the concrete, further enhancing the corrosion process.

One theory on the effect of chloride ions on reinforcement corrosion states that when the chloride concentration becomes large enough, ferrous chloride, or a ferrous chloride complex, is formed on the steel surface of the reinforcing bars. This ferrous chloride replaces the ferric oxide film that was stabilized by the high pH of the cement paste. Since the chlorides are more soluble than oxides, chloride ions move away from the steel, exposing fresh iron to the now corrosive local environment and instigating the electrochemical cell (Perenchio 1994). Chloride ions alone are insufficient to cause corrosion of reinforcing steel (Mehta 1991); in fact, they act more like a catalyst to the corrosion reaction. Adequate moisture and oxygen must also be available.

Carbonation, a reaction between the calcium hydroxide in the hardened cement paste and the carbon dioxide present in the atmosphere, has both a positive and negative impact on the corrosion reaction. Perenchio (1994) has defined carbonation as atmospheric corrosion of the concrete cover. The reaction produces calcium carbonate and water, reducing the pH of the concrete due to the removal of the hydroxyl ions from the pore water solution (Fraczek 1987). Un-carbonated cement paste has a minimum pH of 12.5, but carbonation can reduce this to levels of 8 to 9 (ACI 1992a). This reduction in the pH of the pore solution results in the removal of the protective passive film coating the reinforcing bars, thus contributing to corrosion. However, carbonation can have a positive effect by decreasing concrete permeability, thereby stemming the flow of moisture, oxygen, and chloride ions, and improving the concrete's resistance to corrosion of reinforcement (Emmons 1994). Normally, carbonation is not a problem unless the concrete is cracked or the cover is inadequate (ACI 1989).

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