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A STUDY OF MATERIALS-RELATED DISTRESS (MRD) IN MICHIGAN'S PCC PAVEMENTS—PHASE 2

Final Report

Submitted to the
Michigan Department of Transportation

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16. Abstract

Materials-related distress (MRD) is of concern to the Michigan Department of Transportation, potentially affecting all concrete transportation structures including pavements, bridges, retaining walls, barriers, and abutments. MRD is a direct result of a component breakdown within the concrete matrix due to the interaction between the concrete and its surrounding environment. The specific MRD mechanism and extent varies with location due to differences in local environmental factors, concrete constituent materials, construction practices, deicer applications, and traffic. MRD can occur even in properly constructed PCC pavements having adequate structural capacity, resulting in costly, premature concrete deterioration and eventual failure. This study investigated the occurrence of MRD in Michigan's concrete pavements, using a variety of investigative techniques, including visual assessment, nondestructive deflection testing, strength and permeability testing, microstructural characterization, and chemical methods to determine the causes of observed distress. Based on this investigation, specific recommendations were made regarding treatment of distressed pavements and approaches to avoid the occurrence of these distresses in future concrete pavement construction.

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EXECUTIVE SUMMARY

BACKGROUND

Materials-related distress (MRD) is of concern to the Michigan Department of Transportation, potentially affecting all concrete transportation structures including pavements, bridges, retaining walls, barriers, and abutments. MRD is a direct result of a component breakdown within the concrete matrix due to the interaction between the concrete and its surrounding environment. The specific MRD mechanism and extent varies with location due to differences in local environmental factors, concrete constituent materials, construction practices, deicer applications, and traffic. MRD can occur even in properly constructed PCC pavements having adequate structural capacity, resulting in costly, premature concrete deterioration and eventual failure. This study investigated the occurrence of MRD in Michigan's concrete pavements, using a variety of investigative techniques, including visual assessment, nondestructive deflection testing, strength and permeability testing, microstructural characterization, and chemical methods to determine the causes of observed distress. Based on this investigation, specific recommendations were made regarding treatment of distressed pavements and approaches to avoid the occurrence of these distresses in future concrete pavement construction.

KEY FINDINGS

The following key findings were observed in the course of this study:

- Based on visual inspections of the pavement network, MRD continues to be manifest in in-service pavements throughout the State of Michigan. The manifestations include pattern cracking, scaling, spalling, delamination, joint deterioration, and staining. In the most severe cases, the ability of the pavement to service traffic has been compromised and rehabilitation is needed to restore ride quality. In pavements that have yet to carry the design traffic, this results in unexpected costs associated with premature failure of the structure at great cost to MDOT.
- Structural analysis of pavements affected by MRD, including compressive strength and nondestructive deflection testing, indicates that a slight loss of structural capacity may be incurred as a result of the deterioration. The results are not conclusive, but on average compressive strength and backcalculated PCC modulus of elasticity were each approximately 10 percent less for distress concrete pavements than those obtained on the non-distressed Aggregate Test Road. Additional testing would need to be done to confirm whether this finding is statistically significant.
- Permeability of the concrete was assessed in the field using an air permeability test and in the laboratory using the rapid chloride permeability test. The non-distressed test sites (Test Site Nos. 0 through 4A) comprising the Aggregate Test Road all had RCPT results indicating low permeability. The general trend was increasing permeability based on RCPT results for distressed concrete, although no definitive trend was observed. This is in contrast to the air permeability results that found two of the non-distressed Aggregate Test Road sections (Test Site Nos. 1 and 4A) to be

"permeable." What is interesting is both these sites are constructed with relatively porous coarse aggregate, Test Site No. 1 being slag and Test Site No. 4A being a porous dolomitic limestone. Further, the other test sites constructed with slag also had a "permeable" rating according to air permeability.

- The measured original air contents (ASTM C457) of concrete made with slag coarse aggregate were grossly higher than the specified values, ranging from 9.3 percent to 10.9 percent. This seemingly has had little effect on strength, but it indicates that difficulties in controlling the air content of concrete with slag coarse aggregate exist. Interestingly, one of the slag sites (Test Site No. 19) had the highest measured spacing factor of any of the test sites, even though the total original air content was 10.5 percent.
- Four of the test sites (Test Site Nos. 3, 19, 22, and 26A) had air-void system parameters that are considered marginal for protecting the paste from freeze-thaw damage. Although in no case was distress directly linked to paste freeze-thaw deterioration, the entrained air void system is known to help alleviate pressures generated from other causes such as ASR and sulfate attack.
- Aggregate freeze-thaw deterioration was observed in six test sites (Test Site Nos. 12, 22, 26A, 26B, 27, and 28), all of which were constructed prior to 1975. The deterioration was primarily linked to large, carbonate aggregates from natural gravel sources, and to a lesser degree chert coarse aggregates. It is a testament to the effectiveness of MDOT's policies regarding aggregate screening through the use of MTM 115 that no aggregate freeze-thaw deterioration was observed in more recently constructed pavements
- Alkali-silica reactivity was observed in eight test sites (Test Site Nos. 4, 12, 19, 26A, 26B, 27, 28, and 29). In the case of the Clare Test Road (Test Site Nos. 26A, 26B, and 27), it was associated predominantly with various cherts, impure cherty carbonates, and sandstones coarse aggregate particles that are a constituent of the coarse aggregate. In Test Site Nos. 12 and 22, chert particles present in the natural gravel show signs of being deleteriously reactive, being linked both to alkali-silica reaction product and cracking. In the case of more recently constructed projects, the unique occurrence of ASR in the chert constituent of the fine aggregate in concrete having slag coarse aggregate is most notable. These same chert constituents, which are highly reactive and deleterious in the slag concrete, have been observed to be rather benign in concrete made with natural aggregates. Exactly why this is so is unknown, but enough evidence exists to suggest that a relationship exists between the ASR and the slag coarse aggregate.
- It has been observed that a unique deterioration mechanisms seem to be at work in distressed concrete pavements constructed with slag coarse aggregate (Test Sites Nos. 4, 19, and 29). The deterioration is manifest as a complete breakdown of the concrete matrix, generally in the vicinity of joints. Microscopically, there is strong evidence of calcium sulfide dissolution near the contact zone with the hydrated cement paste, a preponderance of calcium hydroxide in the hydrated cement paste, and secondary ettringite filling adjacent voids and cracks. In addition, sulfate extractions indicate excess sulfate is present in all concrete made with slag coarse aggregate. A hypothesis is emerging that the calcium sulfide that is in contact with the pore solution in the hydrated cement paste is undergoing an oxidation reaction resulting in

dissolution. The calcium is precipitating as calcium hydroxide in the vicinity of the slag particles and the sulfate reacts with aluminate resulting in a type of internal sulfate attack.

- It has been observed that the concrete containing Class F fly ash (Test Site Nos. 0 through 4A) seems to have excellent durability properties. The RCPT results indicate "low" permeability and the chert constituents of the fine aggregate, although known to be reactive, are not involved in deleterious reactions, even in the presence of slag coarse aggregate. Further, although the concrete containing slag coarse aggregate (Test Site No. 1) has excess sulfates and a relatively high rate of ettringite infilling in the air voids, no damaging effect of sulfate attack is observed. It is known that Class F fly ash can help prevent ASR and sulfate attack, and the observations from this study support this conclusion. In contrast, the only test site constructed with Class C fly ash (Test Site No. 4) has undergone rapid deterioration as discussed previously.

RECOMMENDATIONS FOR FUTURE WORK

The following are recommendations for future work based on the results of this study:

- The hypothesis regarding the dissolution of calcium sulfide should be tested. The process is complex with multiple variables impacting the rate at which it might occur, or even if it occurs at all. Obviously, this process is not well enough understood at this time to even clearly implicate the calcium sulfide as being part of the observed deterioration, but enough evidence exist (elevated sulfate contents, observations of evacuated calcium sulfide dendrites, and the presence of considerable amounts of ettringite filling cracks and voids) to support a more in-depth analysis of this hypothesized phenomenon. It is recommended that a controlled laboratory study or studies be initiated to investigate the following:
  - The dissolution process and how it is effected by cement properties and total alkalinity.
  - The relationship between ASR in the chert constituent of the fine aggregate and the presence of slag coarse aggregate. This study must evaluate the effect of chert volume and alkalinity.
  - The ability of fly ash and GBFS to mitigate the effects of calcium sulfide dissolution and ASR in the fine aggregate. Both Class F and Class C fly ashes must be investigated.
- The observations regarding the volume and characteristics of air entrained in concrete containing slag coarse aggregate is interesting. It is possible that the coarseness of the slag particle may lead to more vigorous agitation of the fresh paste during mixing, resulting in increase air void formation. Also, the densified paste region characterized by unhydrated cement grains adjacent to the slag particles should be studied to determine its effect, if any, on the observed deterioration. For example, this might lead to unanticipated shrinkage, both physical and chemical, which in turn could produce cracking at the interface.
- The Aggregate Test Road should continue to be monitored. If signs of MRD emerge, an investigation should ensue to determine the cause of the distress.

- A parametric study of all slag concrete pavements should be conducted using mix design and construction data as well as field inspections. The purpose of this study is to determine if certain mixture design or construction variables have contributed to either a decrease or increase in durability. This limited study has found that Class F fly ash might offer a way to improve the durability of concrete made with slag coarse aggregates, whereas Class C fly ash has had an apparently negative impact. A more detailed large-scale study should be implemented to confirm this finding and determine if other variables are also instrumental.

CHAPTER 1 — INTRODUCTION

Phase I of this multi-phase project has shown that materials-related distress (MRD) is of concern to the Michigan Department of Transportation (MDOT), potentially affecting a significant number of concrete pavements throughout the State (Hiller et al. 1998). MRD is a direct result of deleterious interaction(s) between the concrete and its surrounding environment. This is in contrast to other types of distress (for example transverse fatigue cracking) that are primarily the result of traffic loading and/or poor construction practices. But it is noted that once a MRD is manifested, traffic may exacerbate the rate of deterioration of that distress. Because of this interaction between materials, traffic, and construction, it is difficult at times to attribute failures to one cause or another without a thorough examination of the mechanisms at work.

What is clear is that when a pavement is affected by a MRD, its ability to carry load is compromised to some degree. At a minimum, in many cases the ride quality is impacted by joint deterioration common with many MRDs, which leads to spalling. In some cases, the MRD may be more structurally significant such as when the slab integrity itself is compromised. In either situation, the ability of the pavement to provide a high level of serviceability for the duration of the design life can be negatively affected, necessitating premature rehabilitation at significant cost.

As a result, MDOT initiated this Phase 2 study to better understand the types of MRDs affecting concrete pavements in Michigan, determine suitable treatment strategies, and to prevent the occurrence of these MRDs in future pavement construction. The following is a summary of the project objectives and research plan.

PROJECT OBJECTIVES AND RESEARCH PLAN

The objectives of this project, as stated in the proposal, are as follows:

- Identify and inspect test sites based on the criteria approved in Phase 1.
- Extract sample cores for analysis and diagnosis.
- Conduct laboratory testing to assess relevant material properties.
- Identify the types of MRD affecting concrete pavements in Michigan and suggest treatment alternatives for pavements with MRD.
- Provide recommendations for prevention of MRD in future projects.

As this study is the second phase of a multi-phase research effort, it is a continuation of the Phase 1 study. The four tasks devised to achieve the project objectives, as described below, reflect this continuation.

Task 1: Field Investigations

Task 1 was broken into four sub-tasks that encompassed the selection of test sites, visual assessment of the selected sites, obtaining of specimens for laboratory analysis, and limited non-destructive deflection and air permeability testing.

Task 1a: Selection of Test Sites

The research team sought to identify approximately 30 potential project test sites, representing different locations, materials, and distress manifestations. The test site selection criteria used are listed below.

- Observed pavement distress
- Coarse aggregate type
  - Natural (limestone, glacial gravel)
  - Manufactured (slag, recycled)
  - Grading
- Cementitious materials
- Pavement age
- Geographic location
- Traffic volumes (low,moderate, high)

In total, 32 test sites were initially selected for investigation. These test sites are discussed in Chapter 3 of this final report.

Task 1b: Visual Assessment

Of the 32 test sites selected in Task 1a, 22 were ultimately surveyed using the detailed procedures described in the Phase 1 final report (Hiller et al. 1998). The visual inspection procedures began by driving over the entire project length to judge overall condition. One to two150-m long sites within each project were then selected and a detailed visual inspections conducted. As described, the detailed inspection is similar to a Long-Term Pavement Performance (LTPP) survey (SHRP 1993), but modified specifically to identify MRD distress. The results of the visual assessment are presented in Chapter 3 of this final report.

Task 1c: Coring

Concrete cores were extracted from at 14 of the 22 test sites that were visually assessed. Testing on the cores included compressive strength, rapid chloride ion permeability, and petrographic analysis.

Task 1d: Nondestructive Deflection Testing and Air Permeability

Nondestructive deflection testing (NDT) was conducted on all 14 of the test sites that were cored. The results of this testing provided the backcalculation results for the concrete modulus of elasticity (E_{PCC}) and modulus of subgrade reaction (k-value), as well as load transfer efficiencies. Air permeability testing was also conducted on all 14 test sites.

Task 2: Laboratory Analysis

Task 2, the laboratory analysis, was divided into two sub-tasks as described below.

Task 2a: Strength and Permeability Testing

The extracted field samples were tested to assess the compressive strength (ASTM C 39) and permeability characteristics (AASHTO T 277) of the concrete. The results of the laboratory strength and permeability testing are presented in Chapter 4 of this final report.

Task 2b: MRD Identification

The identification of MRD was conducted in accordance with the guidelines developed for the FHWA which predominately depends upon stereo and petrographic microscopy (Van Dam et al. 2002a). Additional techniques, including scanning electron microscopy and wet-chemistry methods, were also employed as needed. The results of this analysis are presented in Chapter 4 of this final report.

Task 3: Data Analysis and Preparation of Recommendations

The data collected was analyzed to determine the type and extent of MRD observed in the projects evaluated. The results of this data analysis are presented in Chapter 5 of this final report. Based on the analysis, recommendations are made to assist MDOT in material selection, mix design, and in specifying construction practices to reduce or prevent the occurrence of MRD in future MDOT projects. Recommendations regarding remedial treatments are also be made. These recommendations are contained in Chapter 6 of this final report.

Task 4: Final Report

The final task completed in this project was the production of this final report, which includes a full description of the data collection effort, data analysis, recommendations, and conclusion. This final report is divided into the following seven chapters:

Chapter 1 – Introduction
Chapter 2 – Background
Chapter 3 – Test Sites and Field Data Collection
Chapter 4 – Laboratory Testing
Chapter 5 – Data Analysis and Interpretation
Chapter 6 – Treatment and Prevention of Observed MRD
Chapter 7 – Conclusions and Recommendations

CHAPTER 2.0 — BACKGROUND

This chapter provides basic background information regarding the characteristics of MRD in concrete pavements, the techniques used to analyze distressed concrete, and how data is interpreted with the purpose of making a diagnosis.

MRD IN PCC PAVEMENTS

A thorough review of the literature regarding MRD in concrete pavements was presented in the Phase I final report (Hiller et al. 1998) and in reports recent prepared by members of the project team for the Federal Highway Administration (Van Dam et al. 2002a; Van Dam et al. 2002b). For the sake of brevity, this information will not be duplicated here; instead, a short summary is provided that is largely based on the summary contained in the FHWA guidelines (Van Dam et al. 2002a).

The performance of PCC pavements can be adversely affected by the concrete's inability to maintain its integrity in the environment in which it was placed. This loss of integrity, commonly referred to as a lack of durability, occurs even when the structural design is sound. The distresses are generally manifested as cracking or other degradation of the concrete such as scaling or spalling, and are often accompanied by some type of staining or exudate. Depending upon the type of problem and the environment to which the pavement is exposed, these distresses can occur as soon as the first few years following construction of the pavement. Severe climatic conditions (such as freeze-thaw cycling and excess moisture levels, etc.) generally play a significant role in the development of the MRD.

MRD has received greater attention in recent years, in part because of an increased recognition of the importance of durability in the long-term performance of concrete pavements. Contrary to popular belief, durability is not an intrinsic property of the concrete. Instead, "durable concrete is concrete that in the particular environment of service resists the forces in that environment that tend to cause it to disintegrate…" [Transportation Research Board (TRB) 1999]. Whereas previously it was assumed that concrete possessing sufficient strength would inherently be durable, it is now recognized that concrete mixtures must be designed for both strength and durability. In addition, advancements in and more widespread use of sophisticated equipment for examining the microstructure of concrete has increased the ability to accurately identify the mechanisms of MRD. As a result, there is a greater awareness in the pavement industry that concrete is not an inert material, and that care must be exercised when selecting materials and during construction to avoid durability problems.

There are several different types of MRD, and subsets of each in some cases. The occurrence of MRD in a particular pavement is a function of many factors, including the constituent materials (aggregate, cement, admixtures, etc.) and their proportions, the pavement's location (maritime or inland), the climatic conditions (temperature, moisture, and their variation) to which it is subjected, and the presence of external aggressive agents (e.g., roadway deicing chemicals). In general, the development of MRD can attributed to either physical or chemical mechanisms, although the two types of mechanisms often act together to bring about the development of distress. Furthermore, distress from multiple causes may develop together, thereby complicating the determination of the exact cause(s) of material failure.

The MRD types presented in Table 1 are grouped as those caused by physical mechanisms and those cause by chemical mechanisms. A description of these distresses is provided below.

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 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 during the transition from water to 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 under pressure 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 freezes, 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. Again, the resistance to this flow generates internal stresses that may damage the concrete.

If an 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 hydraulic and/or osmotic pressures sufficient to fracture the paste can result.

Table 1. Summary of key MRDs that affect concrete pavements (Van Dam et al. 2002a).

Type of MRD	**Surface Distress Manifestations and Locations**	**Causes/ Mechanisms**	**Time of Appearance**	**Prevention or Reduction**
MRD Due to Physical Mechanisms				
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.
MRD Due to Chemical Mechanisms				
Alkali–Silica Reactivity (ASR)	Map cracking over entire slab area, usually with exudate, 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 to 15 years	Use of nonsusceptible 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 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/cm* 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 to5 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.

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.

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 when the concrete is at or below the freezing point of water. 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 at the concrete surface (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 (Muethel 1997). This occurrence is commonly observed as staining and deterioration in the vicinity of joints and/or cracks.

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). 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 564 lb/yd^3 (335 kg/m^3), limiting the water–cement ratio (*w/cm*) 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]. Recommendation regarding a minimum cement content have recently been disputed by some who argue no minimum cement content requirement is needed (TRB 1999).

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 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 aggregates that fracture due to freezing and thawing, whether gravels or from quarried sources, 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 the magnitude of 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 Na_2O plus 0.658 times the percent K_2O) 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 [American Concrete Institute (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). The longitudinal surface cracks are often more predominate than transverse cracks in pavements due to the lack of restraint along the pavement edge. ASR can also lead to internal horizontal cracks at greater depths within the slab. Upon continued expansion, joint closing, 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 sufficient moisture is available for absorption by ASR gel, and research has shown that pavement concrete in all climates likely will maintain internal humidity levels that 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 alkalies (e.g., chemical deicers), repeated cycles of wetting and drying, and temperature (Farny and Kosmatka 1997; ACI 1998). A more detailed description of the mechanisms resulting in ASR is provided by Helmuth (1993).

The ASR 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) 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, 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 and mineralogical composition, and dosage rate (Farny and Kosmatka 1997). Other types of finely divided materials can also be effective in controlling ASR including ground blast furnace slag (GBFS), silica fume, and natural pozzolans (ACI 1998). More recently, the addition of ASR-inhibiting compounds (e.g., lithium nitrate) has been shown to be effective on highly reactive aggregates (Stark et al. 1993, AASHTO 2002).

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

Alkali–Carbonate Reactivity (ACR)

Alkali–carbonate reactivity (ACR) is another distress caused by undesirable chemical reactions between the concrete 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 dedolomitization 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/or 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 increases 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 the alkali–carbonate reaction (Farny and Kosmatka 1997; ACI 1998).

External Sulfate Attack (ESA) can occur due to the penetration of external sulfate ions (present in groundwater, soil, deicing chemicals, etc.) into the concrete. Although the mechanism of external 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 2) the formation of ettringite through the combination of sulfate ions and hydrated calcium aluminate (ACI 1992). In either case, the formation of the reaction product leads to an increase in solid volume. In hardened paste, the expansive pressures exerted, especially 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/cm* less than 0.45 will help mitigate external sulfate attack (ACI 1992).

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 due to 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 1996):

Ettringite—a high-sulfate calcium sulfoaluminate mineral ($3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$).

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.

An argument can be made 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 is possible if the concrete 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 a sufficient air-void system that has been affected 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.

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 (research suggests a minimum temperature of 65°C to 80°C, with many citing 70°C [Scrivner 1996; Thaulow et al. 1996; Klemm and Miller 1999]), 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.

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 a uniform separation of the paste from the aggregate particles of similar size along the interface. This gap commonly fills with secondary ettringite.

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 and Lewis 1999). The extent of the problem in pavements appears to be small 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.

<u>Corrosion of Embedded Steel</u> appears as rust colored staining, spalling, cracking, and associated deterioration of the concrete above or surrounding the areas affected by active corrosion. Steel corrosion is accelerated in the presence of chloride ions (which can come from a calcium chloride accelerator added to the mix, deicing salts, or seawater) that penetrate the concrete and break down the passivity film that protects embedded steel from corrosion.

Corrosion of embedded steel is controlled by minimizing the permeability of the concrete, providing adequate concrete cover, coating steel with a protective layer, applying a protective coating on the concrete surface to prevent penetration of chlorides and moisture, and suppressing the electrochemical process at the steel surface.

TECHNIQUES FOR EVALUATING DETERIORATED CONCRETE

The techniques used to evaluate concrete suspected of suffering MRD are based upon the use of microscopy and analytical chemistry methods. Recently, the Michigan Tech investigators completed a study on this subject for the FHWA, developing guidelines for use in evaluating distressed concrete pavements (Van Dam et al. 2002a). The following discussion draws upon that body of work.

A key step in identifying the cause of a MRD 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, or engineer is often required to avoid incorrectly diagnosing a problem.

When MRD is suspected of playing a role in the premature deterioration of concrete, laboratory tests are essential 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. These methods include optical microscopy (OM), scanning electron microscopy (SEM), analytical chemistry, and x-ray diffraction (XRD).

Optical microscopy using the stereo OM and the petrographic OM 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). Analytical chemistry plays a very important role in determining key parameters of the concrete (e.g., *w/cm*, chloride content). XRD is applied in some cases but is not widely used in the analysis of 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 many cases of concrete distress are difficult to attribute to only one mechanism, a majority of MRD cases are 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/cm*. 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 MRD 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. 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 conditions or physical properties of the concrete that will assist in diagnosis. After evaluating or assessing the specimen visually, a hand lens or stereo OM can be used to look more closely at pertinent features. In some cases, as a result of this visual and stereo OM 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.

The process is as follows. A sample of concrete exhibiting distress comes into the lab and is first visually inspected. After specimens are produced from a core sample, it is common to use the stereo OM for initial optical analysis and staining techniques to help identify ASR or sulfate phases. Next, the specimen can be viewed in the petrographic microscope and/or SEM. This process of using the stereo OM, petrographic OM, and SEM is iterative and it is not uncommon 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.

A trained petrographer using the petrographic microscope can 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, requirements that necessitate that a highly skilled concrete petrographer conduct the analysis and interpretation. In contrast, the SEM is an instrument 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.

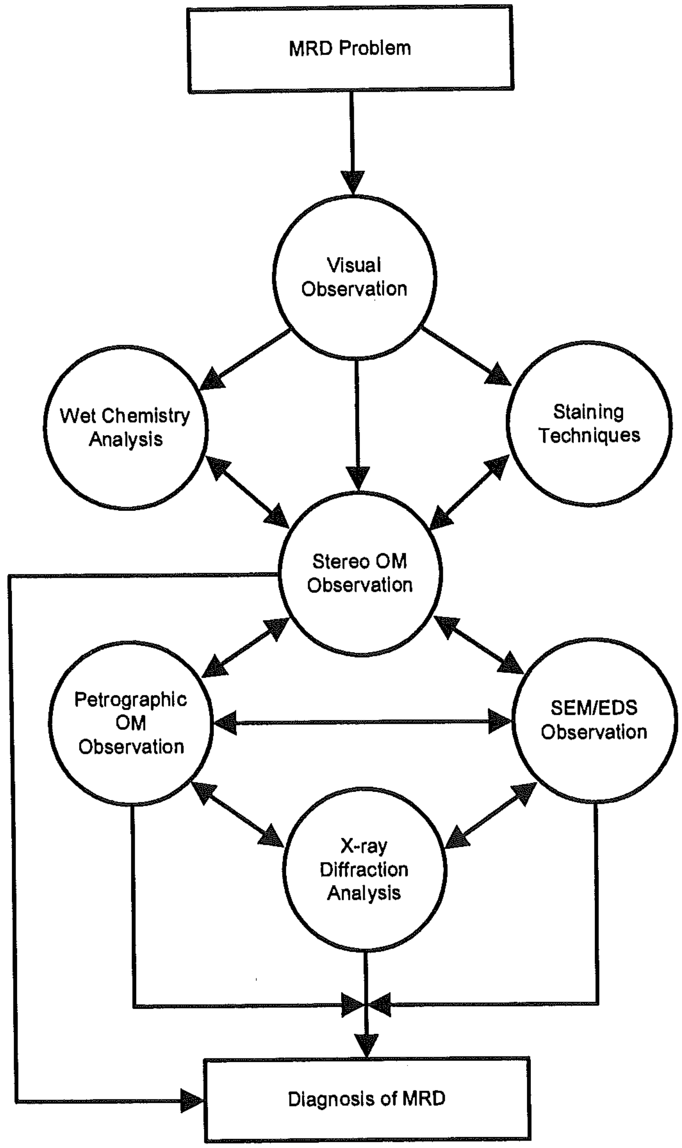


Figure 1. Fundamental process for analyzing a concrete MRD sample.

However, the use of the SEM has some disadvantages. Cracking problems in the conventional SEM (CSEM), microanalysis problems in the environmental SEM (ESEM), and a much higher initial cost with a significant ongoing maintenance cost are 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. 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.

In general, it is recommended that tests be carried out in the order presented in Figure 1. However, if properly done, deviations from this approach should still lead to the same result. That is to say no one test depends directly upon the outcome of another. In practice, some tests must be performed prior to others. One example is the Strategic Highway Research Program (SHRP) uranyl acetate test, which contaminates the concrete with uranium. When using an SEM equipped with an EDS for microanalysis of stained aggregate reaction products, the uranium M-*series* x-ray lines obliterate the potassium K-*series* x-ray lines, making identification of potassium in the aggregate reaction product impossible. Therefore, qualitative EDS microanalysis of ASR reaction products must be performed on unstained concrete. As another example, concrete specimens analyzed in a CSEM are subjected to a high vacuum, and thus microcracking due to desiccation of the paste will occur. It is therefore best to first observe such specimens optically prior to CSEM evaluation to assess paste microcracking.

The diagnosis of MRD in portland cement concrete (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, most cases of concrete distress occur as the result of multiple distress mechanisms. As a result, 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. Most notable of these is the ability to perform a chemical analysis of phases within the concrete. This allows for the absolute identification of reaction and hydration products, greatly facilitating MRD identification.

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 the short 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.

Detailed descriptions of the various test methods and recommended approach to the concrete evaluation can be found in the FHWA guidelines (Van Dam et al. 2002a).

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 microstructure features indicating ASR?” 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 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.

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 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, all possible MRD mechanisms should be listed and assign a relative rating as to the likeliness 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, and highly probable. In this way, the engineers and other interested personnel can get a better understanding of what the analyst 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.

CHAPTER 3.0 — THE TEST SITES AND FIELD DATA COLLECTION

This chapter describes the test sites evaluated in this Phase 2 study and presents the data collected in the field. It includes background on the initial site selection process, the final site selection process, presents the inventory data gleaned from MDOT's records, describes the results of the visual assessment, presents the FWD and air permeability test results, and discusses the coring operations.

INITIAL SITE SELECTION PROCESS

The first phase of this study focused on assessing whether MRD problems were of sufficient severity and extent in Michigan's concrete pavements to warrant detailed evaluation. The initial approach was to use the processed results of the automated data collection effort conducted as part of MDOT's pavement management system (PMS) monitoring. A commercial service provider surveys the entire MDOT pavement network, which consists of 11,980 directional lane miles, on a two-year cycle (one half the network is surveyed each year). Single lane width coverage is obtained by two video cameras mounted on the roof of a van traveling at highway speeds. MDOT personnel, using specially designed workstations, manually interpret the digital images and record the observations directly into the PMS database.

Although this information is extremely useful to MDOT for the management of its pavement network, it was found to be lacking when attempts were made to determine the extent of MRD that was affecting the pavement network. One difficulty was that the distresses identified for the purpose of managing pavements are not specifically differentiated from the type of distress manifestations common with MRD. For example, spalling is a common PCC distress that is recorded in the PMS database. Spalling can be caused by loading and/or environmental conditions that are not related to an MRD, yet many MRD manifestations include spalling. When using the PMS database to search for spalling, a large number of pavements were identified that had spalling that upon more detailed evaluation was found not to be related to MRD. This was also found to be true of map cracking, which may be caused by poor curing practices, but is also a common manifestation of numerous MRD types including alkali-silica reactivity. As a result, it was impossible to determine whether the distress listed in the PMS database was caused by an MRD simply by reviewing the interpreted data.

A second limitation was the inability to easily discern subtle distress manifestations characteristic of the early stages of MRD on the video images. Such occurrences as staining, exudate in cracks, and fine hairline cracking were almost never recorded in the database, even though visual inspections of the pavements revealed that indeed these manifestations existed. As a result, many pavements that had no MRD manifestations recorded in the PMS database turned out to have such manifestations upon manual visual inspections. These two limitations resulted in the conclusion that it was necessary to visually assess a large portion of the concrete pavement network in order to accurately grasp the extent of the problem.

Two teams of researchers, one from Michigan State University and one from Michigan Technological University, manually surveyed approximately 50 percent of the 3,215 miles of concrete surfaced pavement under MDOT's jurisdiction. The surveys entailed driving the pavement sections at the posted speed limit, and then retracing the route while stopping at observed points of interest. These surveys were conducted only during daylight hours to facilitate visibility and safety. Pavements having manifestations consistent with MRD were photographed and notes taken regarding location, extent, and the type of distress observed. Examples of observed distress are presented in Figures 2 through 5. A list of all the sites including all surveyed and non-surveyed sites are presented in Table 2.

It is noted that the survey was not conducted to make an absolute assessment of the extent of MRD throughout the pavement network, but only to determine if distress manifestations consistent with MRD were prevalent throughout the State. It was found that 27 routes were significantly affected (greater than 20 percent of the surface area) by manifestations consistent with MRD. This finding confirmed that MRD continues to be a widespread problem in Michigan's concrete pavements, being recorded from the southeastern corner of the Lower Peninsula to the western Upper Peninsula. Further, the manifestations varied greatly from cracking, staining, and spalling isolated to joints and cracks to map cracking over the entire pavement surface. Surface scaling and corrosion of embedded steel were also noted. These observations suggested that more than one distress mechanism was likely at work. As a result of this initial investigation, it was decided that a more in-depth study was warranted to determine the actual distresses that are at work.

FINAL SITE SELECTION PROCESS

Of the pavements inspected in Phase I, twenty sites were identified for further analysis, comprised of in-service jointed concrete pavements located throughout the state. It was decided that core sampling and laboratory testing would be done on fourteen of these sites using the following selection criteria to obtain a broadly representative group of pavements for study:

- Aggregate type
- Natural (limestone, glacial gravel)
- Manufactured (slag, recycled)
- Environmental conditions (Upper Peninsula, Lower Peninsula)
- Construction process
- Base type (OGDC, dense graded)
- Observed distress manifestations
- Control sections without any visible MRD

Table 3 presents a list of all final test sites selected, indicating those that have been surveyed, cored and tested with the FWD.

Table 2. Information on initial list of test sites.

Site Number	Control Section	Region	County	Location	Year of Const.	Surveyed	Cored	FWD Tested
0	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 432+27; South of MP 6; Section A of MDOT Aggregate Study	1992	x	x	x
1	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 328+13; 54' south of MP 4; Section B of MDOT Aggregate Study	1992	x	x	x
2	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 314+92; 8' south of "Start C" sign; Section C of MDOT Aggregate Study	1992	x	x	x
3	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 243+69; Section D of MDOT Aggregate Study	1992	x	x	x
4A	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 176+36; Right at MP 1; Section E of MDOT Aggregate Study	1992	x	x	x
4	25031	Bay	Genesee	Route: US-23 SB; Starts @ Sta 610+56; South of Hill Road; Between MP 89 & 90	1992	x	x	x
5	80024	Southwest	Van Buren	Route: I-94 EB or WB	1987			
6	44044	Bay	Lapeer	Route: I-69 EB; Starts @ Sta 1534+93; Between MP 154 & 155; Just East of Turnaround; East of Exit 153	1970	x		
7	23061	University	Eaton	Route: I-69 NB or SB	1972			
8	13073	Southwest	Calhoun	Route: I-69 NB or SB	1968			
9	16091	North	Cheyboygan	Route: I-75 EB or WB	1962			
10	47013	University	Livingston	Route: US-23 NB or SB	1960			
11	18033	Bay	Clare	Route: US-27 NB; Starts @ Sta 521+16; Past Rest Area as well as ramp for West 10 to 115; Past MP 179; 13' north of "27 North" sign	1961	x		
12	29011	Bay	Gratiot	Route: US-27 SB; Starts @ Sta 843+45 (stationing in passing lane); Next to North Star Golf Course (next to 1st green and pond); across street from 4143 US-27 NB (white house with barn); Site ends a Rainbow Lake billboard, before Hayes Road and North Star driving range	1957	x	x	x

Table 2(continued). Information on initial list of test sites.

Site Number	Control Section	Region	County	Location	Year of Const.	Surveyed	Cored	FWD Tested
13	59045	Grand	Montcalm	Route: M-46 EB; Starts @ Sta 170+18; West of Edmore; East of "Reduced Speed 40 mph" sign; approximately 1/4 mile east of Edmore Auto Sales; One lane each way, no turn lane for traffic control purposes, fairly wide shoulder	1966	x		
14	79062	Bay	Tuscola	Route: M-81 EB; No station numbering; East of Caro; Starts 160' East of intersection of M-81 and Mcgregory/Lazell Road; Approximately 4.7 miles of M-81/M-24 intersection in Caro; One lane each way, no turn lane for traffic control purposes, gravel shoulder on rising grade	1946	x		
15	23041	University	Eaton	Route: M-43 EB	1949/1947 /1959			
16	76011	University	Shiawassee	Route: M-52 NB	1969			
17	70014	Grand	Ottawa	Route: US-31 SB; South of Grand Haven; Just south of Hayes Street Light; 32' south of turnaround sign "Hayes St East, North 31"; Next to "Michigan Apples 4 Miles on Right at Light" sign; No station numbers (diamond ground) or MP numbers	1954/1956	x		
18	3034	Southwest	Allegan	Route: I-196 SB; South of Holland; Just south of MP 42; Starts @ Station 517+54; Next to "Saugatuck Dunes State Park" sign	1963/1964	x		
19	50011	Metro	Macomb	Route: M-53 NB @station 520. Between 17th and 18th Mile Road and Just in front of Total Gas Station	1988/1989	x	x	x
20	52042	Superior	Marquette	Route: US-41 NB				
21	2041	Superior	Alger	Route: M-28 EB	1961/1966			
22	49025	Superior	Mackinac	Route: I-75 NB, North of Mackinac Bridge, North of MP 349 Station @277+21	Varied (1957)	x	x	x

Table 2(continued). Information on initial list of test sites.

Site Number	Control Section	Region	County	Location	Year of Const.	Surveyed	Cored	FWD Tested
23	21025	Superior	Delta	Route: US-2 EB or WB	1971			
24	21051	Superior	Delta	Route: US-41 NB	1984			
25	22021	Superior	Dickinson	Route: US-2 EB or WB	1976/1979			
26A	18024	Bay	Clare	Route: US-10 WB; Northwest of Clare; Past Old State Road; Section No. 10 of Clare Test Road; Before blue "Next Right" sign; Starts at Station 137+08	1975	x	x	x
26B	18024	Bay	Clare	Route: US-10 WB; Northwest of Clare; Past Old State Road; Section No. 10 of Clare Test Road; Before blue "Next Right" sign; Starts at Station 137+08	1975	x	x	x
27	18024	Bay	Clare	Route: US-10 WB; Northwest of Clare; Past Old State Road; Section No. 11 of Clare Test Road; Past blue "Next Right" sign; Starts at Station 114+00	1975	x	x	x
28	37013	Bay	Isabella	Route: US-27 SB; Starts @ Sta 519+07 (stationing in passing lane); Next to Mt. Pleasant Exit, between Mile post 155 and 156.	1960	x	x	x
29	82291	Metro	Wayne	I-275 SB; at the end of the exit ramp no. 11A on I-275 HEADING TOWARDS SOUTH (Toledo). The road ends at the Willow Metropark and the exit sign says towards East Willow Road.	1976	x	x	x

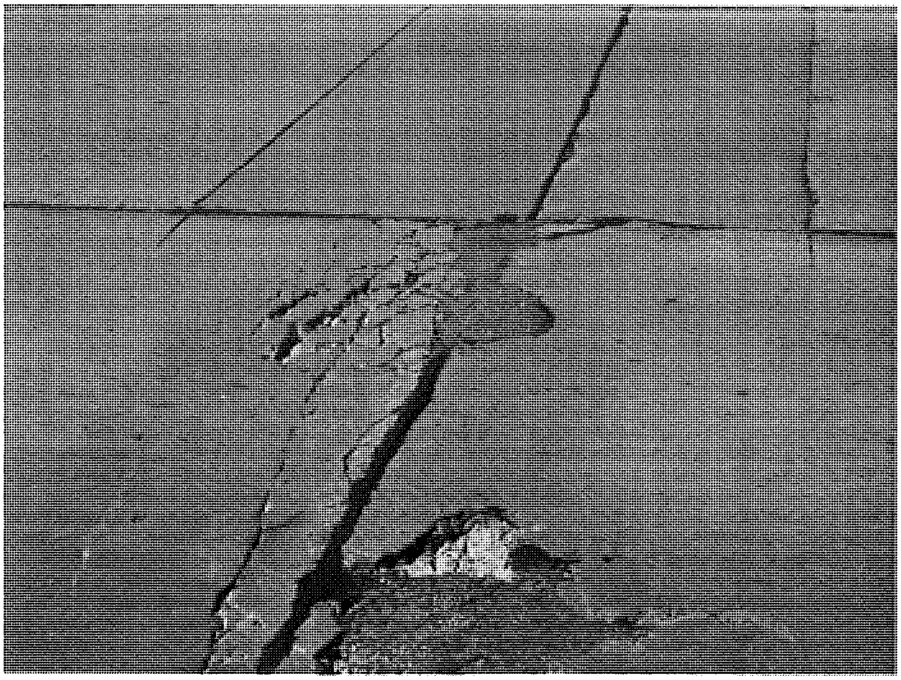


Figure 2. Cracking, staining, and spalling at joint.

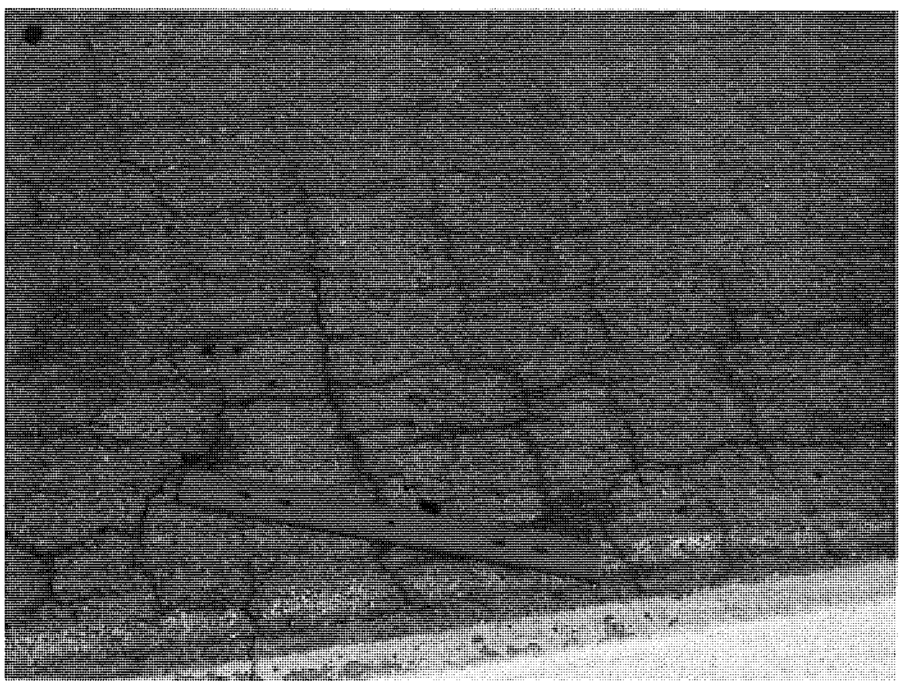


Figure 3. Map cracking with discoloration.

Figure 4. Corrosion of embedded steel.

Figure 5. Surface cracking with exudate.

Table 3. Final test sites selected for detailed evaluation.

Site Number	Control Section	Region	County	Location	Year of Const.	Surveyed	Cored	FWD Tested
0	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 432+27; South of MP 6; Section A of MDOT Aggregate Study	1992	x	x	x
1	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 328+13; 54' south of MP 4; Section B of MDOT Aggregate Study	1992	x	x	x
2	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 314+92; 8' south of "Start C" sign; Section C of MDOT Aggregate Study	1992	x	x	x
3	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 243+69; Section D of MDOT Aggregate Study	1992	x	x	x
4A	58034	University	Monroe	Route: US-23 SB; Starts @ Sta 176+36; Right at MP 1; Section E of MDOT Aggregate Study	1992	x	x	x
4	25031	Bay	Genesee	Route: US-23 SB; Starts @ Sta 610+56; South of Hill Road; Between MP 89 & 90	1992	x	x	x
12	29011	Bay	Gratiot	Route: US-27 SB; Starts @ Sta 843+45 (stationing in passing lane); Next to North Star Golf Course (next to 1st green and pond); across street from 4143 US-27 NB (white house with barn); Site ends a Rainbow Lake billboard, before Hayes Road and North Star driving range	1957	x	x	x
19	50011	Metro	Macomb	Route: M-53 NB @station 520. Between 17th and 18th Mile Road and Just in front of Total Gas Station	1988/1989	x	x	x
22	49025	Superior	Mackinac	Route: I-75 NB, North of Mackinac Bridge, North of MP 349 Station @277+21	Varied (1957)	x	x	x
26A	18024	Bay	Clare	Route: US-10 WB; Northwest of Clare; Past Old State Road; Section No. 10 of Clare Test Road; Before blue "Next Right" sign; Starts at Station 137+08	1975	x	x	x

Table 3(continued). Final test sites selected for detailed evaluation.

Site Number	Control Section	Region	County	Location	Year of Const.	Surveyed	Cored	FWD Tested
26B	18024	Bay	Clare	Route: US-10 WB; Northwest of Clare; Past Old State Road; Section No. 10 of Clare Test Road; Before blue "Next Right" sign; Starts at Station 137+08	1975	x	x	x
27	18024	Bay	Clare	Route: US-10 WB; Northwest of Clare; Past Old State Road; Section No. 11 of Clare Test Road; Past blue "Next Right" sign; Starts at Station 114+00	1975	x	x	x
28	37013	Bay	Isabella	Route: US-27 SB; Starts @ Sta 519+07 (stationing in passing lane); Next to Mt. Pleasant Exit, between Mile post 155 and 156.	1960	x	x	x
29	82291	Metro	Wayne	I-275 SB; at the end of the exit ramp no. 11A on I-275 heading south (Toledo). The road ends at the Willow Metropark and the exit sign says towards East Willow Road.	1976	x	x	x

INVENTORY DATA

Using the criteria for site selection discussed in the work plan, fourteen sites were selected for coring and FWD testing. The sites selected were constructed of a variety of materials and subjected to varying climatic conditions. An inventory database was compiled for the test sites, including:

- Pavement cross-section
- Joint Spacing
- Shoulder type/width
- Pavement age
- Concrete mixture design data
- Aggregate type in mix design
- Slump during construction
- Air content at the time of construction
- Compressive/flexural strength of the concrete

This information was compiled from the Michigan Department of Transportation project construction and design records. The PCC slab thickness ranged from 220 mm to 270 mm. The test sites ranged from 7 to 43 years in age and were subjected to varying amounts of traffic. The concrete mixture design for all the test sites was in accordance with standard DOT paving mixture specifications. The cement content for all the mixture designs ranged from 230 kg/m^3 to 328 kg/m^3, and the coarse aggregate type included various sources of crushed carbonate (limestone and dolomite), gravel, and blast furnace slag. For a few of the sites, some of the inventory data could not be found in the MDOT records. Type of inventory data, which could not be found in the records includes pavement cross sectional data, mix design data, slump, air content and other data. The information collected is summarized in Tables 4 through 8.

Table 4. Joint spacing, length of the test section, shoulder type, and shoulder width.

Site Number	Number of Slabs	Joint Spacing (m)	Length of Test Section (m)	Shoulder Width (m)	Shoulder Type
0	20	8.2	164	3.0	Concrete
1	20	8.2	164	3.0	Concrete
2	20	8.2	164	3.0	Concrete
3	20	8.2	164	3.0	Concrete
4A	20	8.2	164	3.0	Concrete
4	20	8.2	164	3.0	Concrete
12	6	30.2	181	2.7	Asphalt
19	6	8.2/12.5	218	Nil	N/A
22	6	30.2	181	2.7	Asphalt
26A	20	3.7 / 4.0 / 5.8 / 5.5	95	2.7	Asphalt
26B	20	3.7 / 4.0 / 5.8 / 5.5	95	2.7	Asphalt
27	20	3.7 / 4.0 / 5.8 / 5.5	95	2.7	Asphalt
28	6	30.2	181.2	2.7	Asphalt
29	9	22.55	202.95	2.7	Asphalt

Table 5. Cross-section and base type.

Site Number	Cross-Section			Base Type
	PCC (mm)	Base (mm)	Subbase (mm)	
0	267	178	305	5G Asphalt Stab. O.G.D.C. + 21AA Limestone Agg. Base
1	267	178	305	5G Asphalt Stab. O.G.D.C. + 21AA Limestone Agg. Base
2	267	178	305	5G Asphalt Stab. O.G.D.C. + 21AA Limestone Agg. Base
3	267	178	305	5G Asphalt Stab. O.G.D.C. + 21AA Limestone Agg. Base
4A	267	178	305	5G Asphalt Stab. O.G.D.C. + 21AA Limestone Agg. Base
4	254	178	No Data	Asphalt Stab. O.G.D.C. + Agg. Base
12	229			
19	229	102	254	Aggregate Base
22	229	No Data	No Data	No Data
26A	229	102	254	ATPM
26B	229	102	254	ATPM
27	229	102	254	Aggregate Base
28	229	76.2	356	Aggregate Base
29	229	102	254	Aggregate Base

Table 6. Aggregate type, grade of concrete, and mixture proportions.

Site Number	Type of PCC Agg.	Grade of Concrete	Mix Design Proportions (per cubic meter of concrete)			
			Cement (kg)	Water (kg)	Air (%)	Comments
0	Crushed Limestone	35P	307 (5.5 sacks)	169 (oven dry aggs)	design 5.5%, specified 6.5%, tolerance +/- 1.5%	b/b_o=0.72, Class F Fly Ash=46 kg/m^3 of concrete
1	Blast Furnace Slag	35P	285 (5.1 sacks)	162 (oven dry aggs)	design 5.5%, specified 6.5%, tolerance +/- 1.5%	b/b_o=0.72, Class F Fly Ash=43 kg/m^3 of concrete
2	Natural Gravel	35P	307 (5.5 sacks)	144 (oven dry aggs)	design 5.5%, specified 6.5%, tolerance +/- 1.5%	b/b_o=0.72, Class F Fly Ash=46 kg/m^3 of concrete
3	Crushed Limestone	35P	285 (5.1 sacks)	157 (oven dry aggs)	design 5.5%, specified 6.5%, tolerance +/- 1.5%	b/b_o=0.72, Class F Fly Ash=43 kg/m^3 of concrete
4A	Natural Gravel	35P	307 (5.5 sacks)	150 (oven dry aggs)	design 5.5%, specified 6.5%, tolerance +/- 1.5%	b/b_o=0.72, Class F Fly Ash=46 kg/m^3 of concrete
4	Slag	35P	285 (5.1 sacks)	154.3	design 5.5%	Class C Fly Ash=43 kg/m^3 of concrete
12	Natural Gravel	35P	300 (5.5 sacks)	155.23 (oven dry aggs)	design 5.5%	b/b_o=0.78
19	Slag	35P	328 (6.0 sacks)	147	design 5.5%, tolerance +/- 1.5%	b/b_o=0.78
22	Natural Gravel	35P	301 (5.5 sacks)	162	design 6.0%	b/b_o=0.78
26A	Natural Gravel	35P	252	121	design 6.5%	b/b_o=0.72
26B	Natural Gravel	35P	252	121	design 6.5%	b/b_o=0.72
27	Natural Gravel	35P	252	121	design 6.5%	b/b_o=0.72
28	Natural Gravel	35P	231	108 (Oven dry)	No Data	No Data
29	Blast Furnace Slag	35P	306 (5.6 sacks)	175	Design 6.5%	No Data

Table 7. Coarse and fine aggregate properties.

Site Number	Coarse Aggregate				Fine Aggregate			
	Mass (kg/m³ of concrete)	Class	Pit No.	Other	Mass (kg/m³ of concrete)	Class	Pit No.	Other
0	1003 (Oven dry)	6AA	93-3	AC=2.57%, DRUW=87 pcf, G=2.57	796 (Oven dry)	2NS	30-35	AC=1.71%, G=2.59
1	830 (Oven dry)	6AA	82-22	AC=3.34%, DRUW=72 pcf, G=2.29	918 (Oven dry)	2NS	30-35	AC=1.71%, G=2.59
2	1211 (Oven dry)	6A	30-35	AC=0.86%, DRUW=105 pcf, G=2.71	670 (Oven dry)	2NS	30-35	AC=1.71%, G=2.59
3	1026 (Oven dry)	6AA	58-8	AC=2.64%, DRUW=89 pcf, G=2.60	843 (Oven dry)	2NS	30-35	AC=1.71%, G=2.59
4A	1165 (Oven dry)	6A	63-97	AC=1.24%, DRUW=101 pcf, G=2.66	689 (Oven dry)	2NS	30-35	AC=1.71%, G=2.59
4	859 (Oven dry)	6AA	82-19		971 (Oven dry)	2NS	63-54	
12	1437	4A/10A	34-36	DRUW=102 pcf, MC=1.7%, G=2.75/2.67, AC=0.74/1.74	718	2NS	34-36	MC=7.4%, AC=0.81
19	1274	6A/4A	No Data	MC=2.0%/2.5%, AC=1.24%/0.84 %, G=2.67/2.71, DRUW=102 PCF	649	2NS	No Data	AC=1.42%, MC=6.7%, G=2.62
22	1260	4A/10A	49-44	MC=0.9%/2.3%, AC=1.17%/1.49 %, G=2.70/2.69	644	2NS	49-44	AC=0.83%, G=2.68
26A	1158	6A	67-2	AC=1.50%, DRUW=102 pcf, G=2.64	629	2NS	67-2	AC=1.02%, G=2.60
26B	1158	6A	67-2	AC=1.50%, DRUW=102 pcf, G=2.64	629	2NS	67-2	AC=1.02%, G=2.60
27	1158	6A	67-2	AC=1.50%, DRUW=102 pcf, G=2.64	629	2NS	67-2	AC=1.02%, G=2.60
28	998	6A/10A	67-2	DRUW=102 pcf, MC_{6A}=1.2%, MC_{10A}=2.5%	478	2NS	67-2	MC=5.6%
29	847	6A	Trenton	DRUW=102 pcf, MC=2.0%	996	2NS	81-57	MC=5.0%

Table 8. Slump, air content, 28-day compressive strength and flexural strength.

Site Number	Slump (mm)	Air Content (%)	28-Day Compressive Strength (mPa)	Flexural Strength (mPa)
0	70	6.3	30.7	28-day 5.16
1	57	7.3	31.9	28-day 4.72
2	64	7.5	30.0	28-day 4.96
3	64	6.4	32.8	28-day 5.05
4A	64	7	30.2	28-day 4.96
4	No Data	No Data	No Data	No Data
12	50.8	5.5	35.3	7 day 4.14/ 14 day 4.66
19	54	7.1	56.3	No Data
22	64	No Data	37.6	7-day 4.31/ 14-day 4.93
26A	32	7	44.4	14-day 4.88
26B	32	7	44.4	14-day 4.88
27	32	7	44.4	14-day 4.88
28	38.1	No Data	46.2	14-day 4.34
29	64	7	No data	28-day-5.61

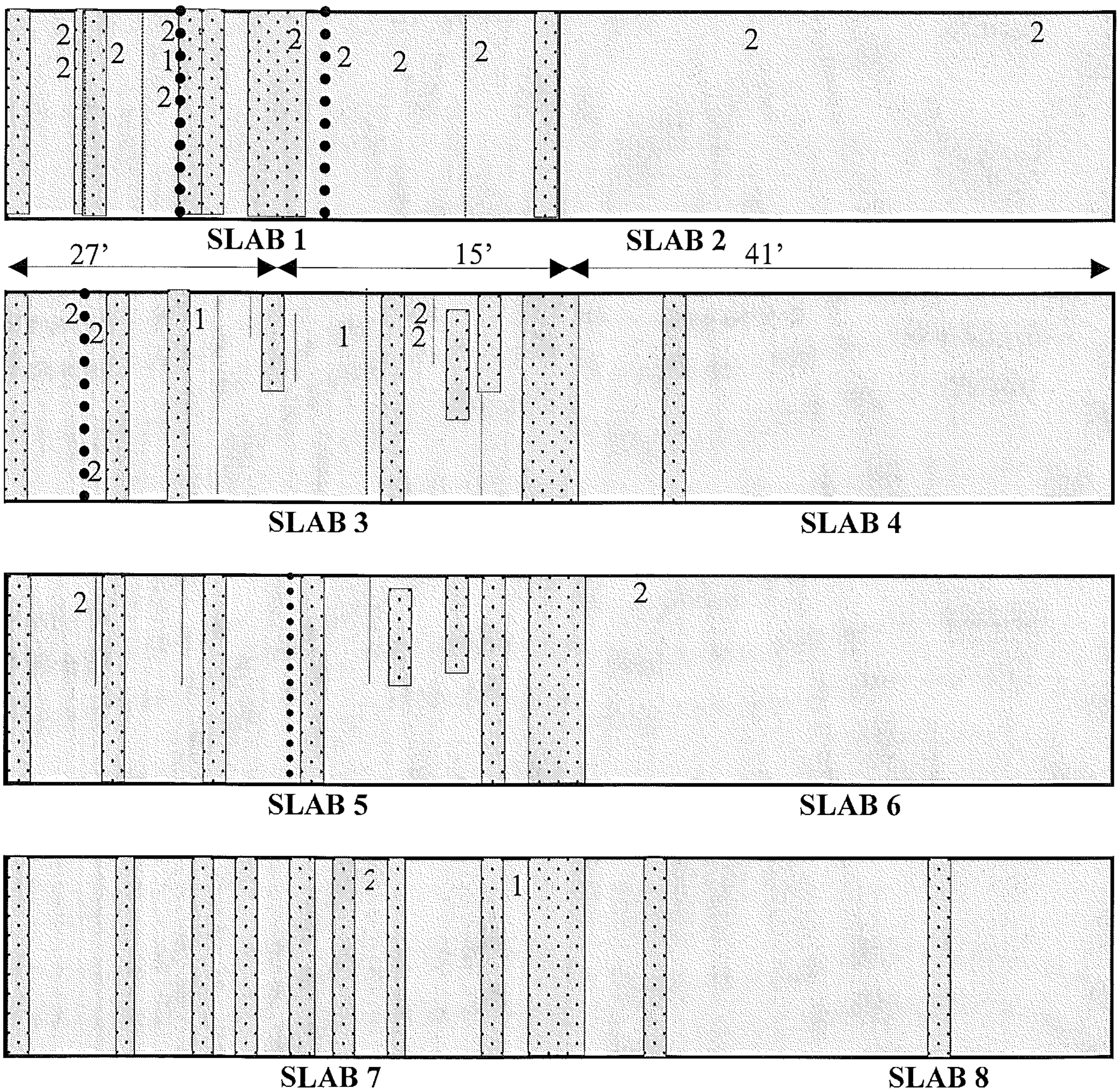
VISUAL ASSESSMENT

At each test site a detailed visual distress survey was conducted to document distress consistent with LTPP distress types (SHRP 1993). Additional characteristics consistent with MRD manifestations were also recorded. The methods used drew heavily from work conducted for the Federal Highway Administration, which has led to the development of guidelines for conducting a field survey and sampling protocol for pavements affected by MRD (Van Dam et al. 2002a). The protocol consists of an initial project survey conducted from a slow moving vehicle in which the overall consistency in distress was assessed over the entire project length. If the distress was uniformly distributed, a single 150-m long representative pavement section was selected and inspected using a detailed procedure in which distress type, extent, and severity are described. Additional features including staining, the presence of exudates, expansion, etc. is also noted. If the distress is not consistent over the project lengths, additional pavement sections were inspected that are representative of each condition level observed. Each test site consisted of approximately 5 to 20 slabs depending on the joint spacing. Figure 6 shows a typical test site layout, including observed distress manifestations, collected in this study. Appendix A provides all the photographs taken regarding the distresses found at all the sites. Appendix B provides all of the test site layouts and the observed distress manifestations collected during this study. Comments regarding the results of the visual assessment for the 14 test sites that were selected for coring are summarized in Table 9.

DATE: **xx/xx/00** **SITE #:** **Y**

LOCATION: Route: XXX; Starts @ Sta 519+07; Between MP 155 & 156; Just North of Entrance Ramp;

JOINT SPACING: 15', 27', 41', asphalt shoulders



KEY:

Transverse Cracking	*Patching*		
Low	Full-depth Concrete	1. Spalling	5. High steel
Moderate	Asphalt Patch	2. D-Cracking	6. Polished agg
High	Expansion joint	3. Poor sealant	7. Faulting
		4. ASR	8. Long. crack

COMMENTS: Most of the transverse cracks are patched with asphalt. Spalling is present on the longitudinal joints (every 6 feet) but has also been patched. Popouts occurred here and there.

Figure 6. Typical test site layout, including observed distress manifestations.

Table 9. Comments on the visual condition of test sites.

Site Number	Distress Comments
0	Shoulder is different concrete type than mainline pavement (different color); Shrinkage cracking typically 6-8" apart and very tight, predominantly transverse; No popouts
1	Small hole in Slab 7; Construction repairs (epoxy) on Slab 15
2	Stain on Crack 1 of Slab 5; Poor construction grooving on entire site; Coring hole on Slab 10; Construction repairs (exopy) on Slabs 13 and 14; Popouts: 2-3 per m^2
3	Joint sealant drop between Slabs 14 and 15; Poor construction grooving on entire site; Coring hole on Slab 9; Possible dowel misalignment cracks on Slabs 2 and 3; Popouts: ~ 1 per m^2
4A	Popouts: 3 per m^2
4	ASR-like cracking in shoulder near joints; ASR-like cracking on entire mainline pavement; All joints sealants are deficient; Longitudinal cracking (~ 6" spacing) in slab due to connecting ASR-like cracks; Popouts: 0.5 per m^2
12	Polished aggregates and tiny cracks (possibly old shrinkage cracking) throughout site; Popouts: ~ 3 per m^2
19	Third point cracking in every 41 feet long slab. ASR type of cracking visible in some of the areas near joints and midslab. There is also D-cracking pattern in some of the slabs.
22	Spalling has occurred along the Longitudinal Joints; D-cracking-type pattern near transverse joints and cracks with white staining in cracks; Popouts: 1 per m^2
26A	Longitudinal joint is completely spalled; D-cracking-type pattern near transverse joints (corner) and cracks with white staining in cracks; Popouts: 1.5 per m^2
26B	Longitudinal joint is completely spalled; D-cracking-type pattern near transverse joints (corner) and cracks with white staining in cracks; Popouts: 1.5 per m^2
27	Longitudinal joint is completely spalled; D-cracking-type pattern near transverse joints (corner) with white staining in cracks; Spallings are more sever than in Sites 26A and B; Popouts: 1.5 per m^2
28	All the joints and most of the Transverse cracks are asphalt patched. D-Cracking type pattern with white staining is visible near all the transverse cracks and some of the joints (corner). Spalling had occurred in Longitudinal Joints almost in every 6 feet but already been asphalt patched. Popouts: 1-2 per m^2. Polished Aggregate in Wheel path.
29	The site seems to be a typical MRD pavement with very less traffic on it. D-Cracking type pattern with white staining is visible near all the transverse cracks and most of the joints. Spalling had occurred in transverse joints, Longitudinal Joints and transverse cracks. Some of the spalls are already in poor conditioned and patched with asphalt mixture.

The test sites investigated had varying amounts of observed distress. Test sites 0, 1, 2, 3, and 4a are from MDOT's Aggregate Test Site, located on southbound US- 23 near the Ohio border. These sections contained very little observable distress and were intentioned to be used as control sections for the study. The five test sites share similar pavement and mixture designs, but the coarse aggegate type was varied to evaluate its effect. Built in 1993, there is little observable distress. The remaining test sites were moderately to heavily distressed, exhibiting not only MRD manifestations but also other distresses attributable to loading and/or climatic effects. Typical MRD manifestations observed included map cracking, "D" cracking, popouts, fine cracking with white exudate, spalling/paste disintegration, corrosion, scaling, and staining on the surface near joints. As mentioned, photographs of each test site are presented in Appendix A and detailed distress characterization is presented in Appendix B.

FALLING WEIGHT DEFLECTOMETER TESTING

Falling weight deflectometer (FWD) testing was performed in an attempt to assess the loss in structural capacity resulting from the MRD. Deflection testing was done in the middle of the slab to enable back-calculation of the concrete elastic modulus (E_{PCC}) and the modulus of subgrade reaction (k-value), using the procedures documented in Smith et al. (1997). Slab thicknesses were determined from extracted cores. An E_{PCC} and k-value was then calculated for the test site by averaging the values determined at the various sensors. Information regarding back-calculation of the k-value is found in Darter et al (1995). The range in back-calculated k-values ranged from 20.8 to 130.5 kPa/mm whereas E_c varied from 2.64×10^4 to 8.99×10^4 mPa. The FWD testing machine, which is owned by Michigan Department of Transportation, is presented in Figure 7.

The back-calculated E_{PCC} of the distressed test slabs was approximately 10 percent lower than that calculated for the control slabs. This finding is important when considering the impact that an MRD has on the structural capacity, and therefore the longevity, of a concrete pavement section. Table 10 provides the back-calculation results for all the sections.

AIR PERMEABILITY TESTING

In-place air permeability testing was also conducted in the field in accordance with SHRP protocol 2031. This is a rapid field test for assessing the in-situ concrete permeability, assisting in the bulk evaluation. The following criteria is used to make a qualitative assessment of the air permeability of the concrete tested using this test method:

Air Flow (mL/minute)	Air Permeability
<5	Good
5-15	Moderate
>16	Permeable

Table 11 presents the air permeability data, whereas Figure 8 is a photograph of the air permeability testing machine.

Figure 7. MDOT's FWD testing at Site 4A, US-23 SB.

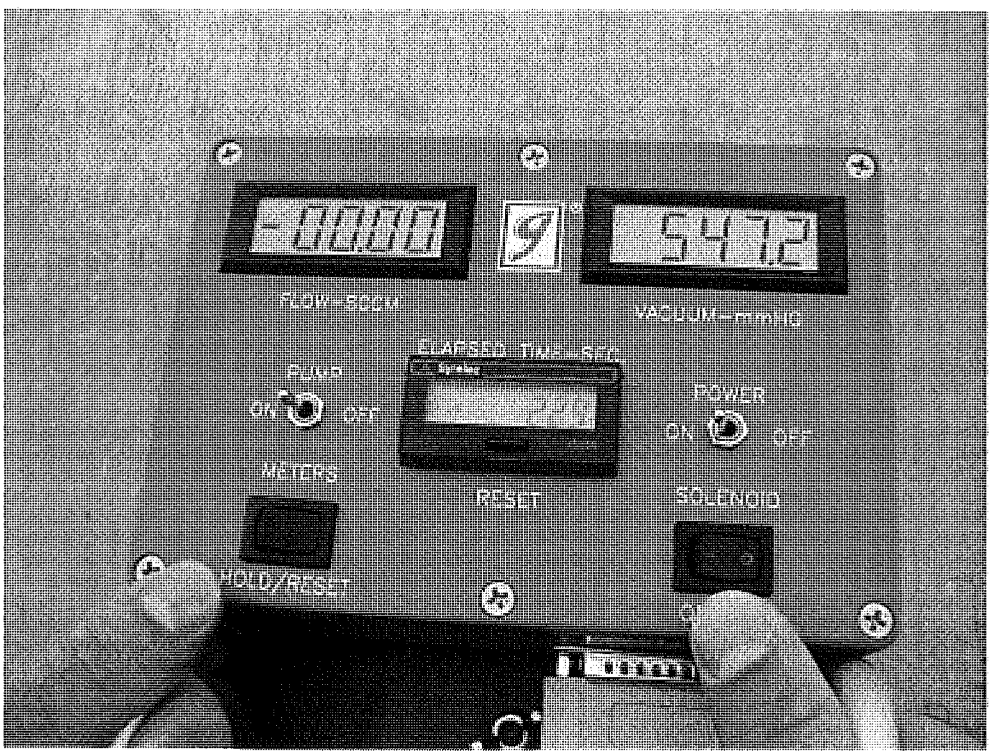


Figure 8. Air permeability apparatus control panel.

Table 10. FWD data and backcalculated k-value and E_{PCC}.

Site Number	Slab No.	Basin Area (mm)	Radius of Relative Stiffness (mm)	k-Value (kPa/mm)	Elastic Modulus of PCC (mPa)
0	3	1,098.4	939.1	93.8	3.77E+04
	4	1,089.2	921.0	98.2	3.64E+04
1	16	1,075.1	894.7	97.4	3.26E+04
	18	1,080.6	904.8	99.9	3.56E+04
2	3	1,255.9	1,380.6	47.9	8.99E+04
	4	1,113.6	970.0	97.5	4.53E+04
3	1	1,082.9	909.1	105.6	3.49E+04
	2	1,091.0	924.6	105.6	3.51E+04
4A	6	1,079.4	902.5	125.9	4.91E+04
	5	1,074.4	893.4	130.5	4.31E+04
4	18	1,098.4	939.1	93.8	5.25E+04
	20	1,089.2	921.0	98.2	4.34E+04
12	2	1,066.7	879.4	69.3	3.62E+04
	2	1,065.3	877.0	74.0	3.53E+04
19	7	1,237.3	1,310.2	20.8	4.12E+04
	2	1,225.3	1,268.1	28.4	4.50E+04
22	1	1,165.4	1,090.7	44.9	5.56E+04
	2	1,131.9	1,009.8	60.9	5.11E+04
26A	7	1,082.4	908.2	62.9	3.62E+04
	9	1,128.2	1,001.6	56.0	4.77E+04
26B	8	1,119.6	982.8	53.4	4.62E+04
	10	1,074.6	893.7	60.4	3.06E+04
27	11	1,148.8	1,049.4	54.9	5.80E+04
	12	1,157.1	1,069.6	52.4	6.06E+04
28	3	1,172.8	1,110.1	30.4	4.02E+04
	2	1,159.6	1,075.8	33.3	3.92E+04
29	1	1,051.3	852.9	54.1	3.24E+04
	5	1,094.6	931.6	46.0	2.92E+04
	7	1,043.6	840.0	50.9	2.64E+04
	9	1,081.7	906.8	48.0	2.89E+04

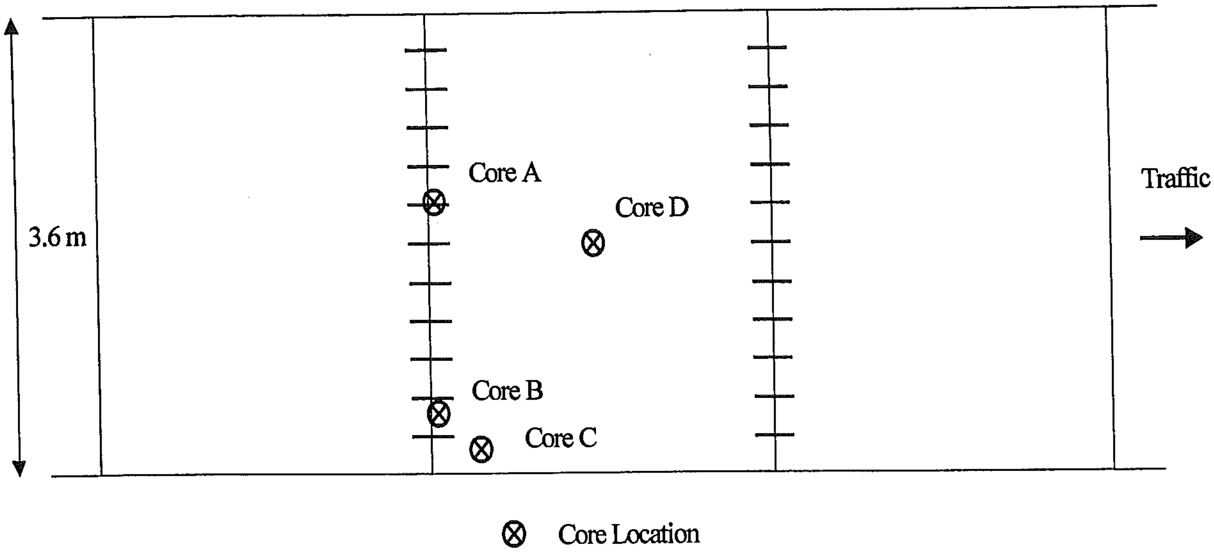
Table 11. Air permeability results from the test sites.

Site Number	Slab No.	Air Permeability (mL/min)
0	3	4.24
	4	2.43
1	16	19.52
	18	13.41
2	4	5
	3	4.21
3	1	7
	2	6.16
4A	3	17.45
	2	19.73
4	20	16.32
	18	16.76
12	2	12.37
	2	13.51
22	2	13.63
	1	13.63
26A	9	13.63
	7	13.63
26B	10	13.64
	8	13.64
27	11	13.64
	12	13.64
28	2	19.233
	3	10.2833
29	1	N/A
	5	10.84
	7	14.732
	9	23.69

CORING

After the visual assessment, FWD testing, and air permeability testing were completed, coring was conducted. Unless unusual distress patterns were encountered, a single slab was cored in each site. The typical coring locations used for obtaining specimens for the petrographic evaluation are shown in Figures 9. Depending on whether the distress is primarily restricted to joint/crack locations or distributed over the entire pavement surface, Figure 9a or 9b is used, respectively. This pattern is consistent with the recommendations presented in the FHWA guidelines (Van Dam et al. 2002a).

Additional cores were extracted for compressive strength tests (in accordance with ASTM C39) and rapid chloride permeability (in accordance with AASHTO T277). For this purpose four 6-inch diameter cores were obtained from two other slabs. One non-distressed/less distressed slab and another distressed slab were chosen apart from the slab sampled previously.



Core Summary

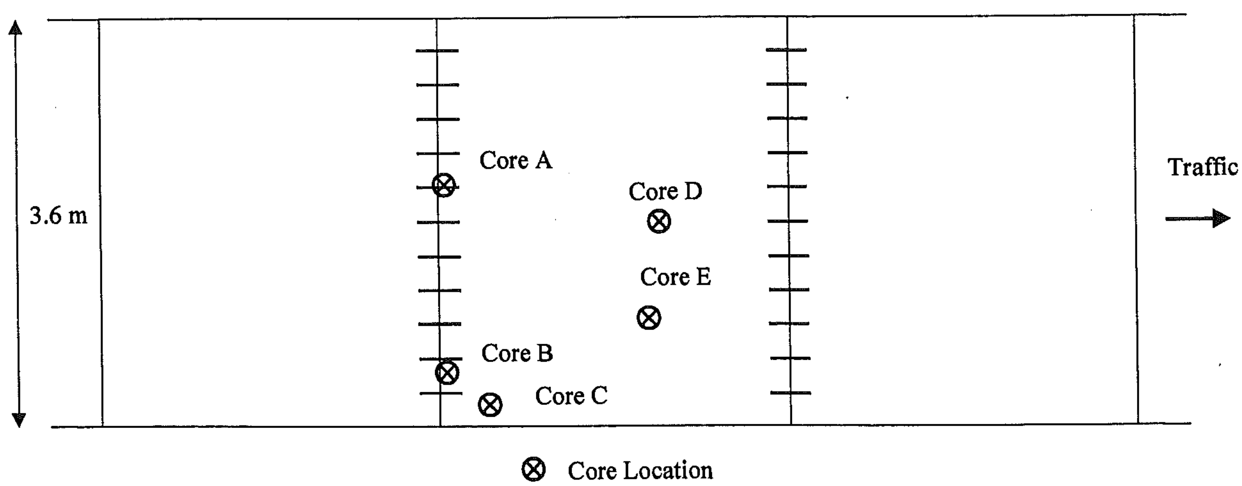
Core A Take across the joint and over a dowel near the middle of the slab, offset about 50 mm from center of joint.

Core B Take across the joint and between dowels, within 1 m of slab edge.

Core C Take about 0.3 m away from joint and 0.3 from slab edge.

Core D Take in the center of the slab in an area free of MRD.

(a)



Core Summary

Core A Take across the joint and over a dowel near the middle of the slab, offset about 50 mm from center of joint.

Core B Take across the joint and between dowels, within 1 m of slab edge.

Core C Take about 0.3 m away from joint and 0.3 from slab edge.

Core D Take in the center of the slab in an area as free of MRD as possible.

Core E Take in the center of the slab in an area containing MRD.

(b)

Figure 9. Core sampling plan for distress concentrated at joints and cracks (a) and distress over entire surface (b) (Van Dam et al. 2002a).

CHAPTER 4.0 — LABORATORY TESTING

Laboratory testing conducted for this study can be divided in mechanical/permeability testing and microstructural analysis. Mechanical/permeability testing included compressive strength (ASTM C39) and rapid chloride permeability testing (AASHTO T277). Microstructural analysis included stereo and petrographic optical microscopy and analytical chemistry. This chapter reports the results of this laboratory testing.

MECHANICAL/PERMEABILITY TESTING

Compressive Strength Testing (ASTM C39)

In general, it was found that concrete cores extracted from "distressed" slabs had lower compressive strengths compared to cores extracted from non-distresses slabs. This finding is intuitive, but still of valuable since it demonstrates how the occurrence of MRD can negatively impact not only the ride quality of the pavement, but also its structural capacity. On average, the distressed cores exhibited compressive strengths that were 10 percent lower than the non-distressed cores. There is insufficient data to support whether this difference is statistically significant. Table 12 presents the results of the compressive strength testing, with the back-calculate E_{PCC} provided for comparison. Average values are presented for each site.

Rapid Chloride Permeability Test (RCPT) Results (AASHTO T277)

Rapid determination of the chloride ion permeability of concrete is determined in the laboratory using AASHTO designation T277-83. This test method covers the determination of the permeability of conventional portland cement concrete. It consists of monitoring the amount of electrical current passed through 95 mm (3.75 inch) diameter by 51 mm (2 inch) long cores when one end of the core is immersed is a sodium chloride solution and a potential difference of 60 volt dc is maintained across the specimen for 6 hours. The total charge passed, in coulombs, is related to chloride permeability. Table 13 provides a relationship between the permeability and the charge passed through the cores (Whiting 1981). Measured rapid chloride permeability ranged from 900 to 9133 coulombs, corresponding to ratings of very low to high permeability, respectively. Figure 10 is a plot between the two measures of permeability.

Both the air and rapid chloride ion permeability test data is provided in Table 14 and plotted in Figure 10. It is observed that sites exhibiting moderate to high levels of MRD had moderate to high permeability ratings. This finding is not surprising, but it is uncertain what the causal relationship is since it is impossible to ascertain whether the high permeability contributed to the MRD or whether the presence of the MRD has resulted in high permeability. It is also noted that there is a weak correspondence between the qualitative air permeability ratings obtained in the field and the laboratory derived rapid chloride ion permeability rating, more so at low permeability levels.

Table 12. Backcalculated E_{PCC} and laboratory determined compressive strength.

Site Number	Slab No.	Back-Calculated Elastic Modulus of PCC (mPa)		Compressive Strength of PCC Cores (kPa)	
		Data	Average for Site	Data	Average for Site
0	3	3.77E+04	3.71E+04	46.165	3.76E+01
	4	3.64E+04		29.090	
1	16	3.26E+04	3.41E+04	32.470	3.79E+01
	18	3.56E+04		43.411	
2	3	8.99E+04	6.76E+04	45.068	3.68E+01
	4	4.53E+04		28.476	
3	1	3.49E+04	3.50E+04	45.616	4.05E+01
	2	3.51E+04		35.433	
4A	6	4.91E+04	4.61E+04	28.146	2.99E+01
	5	4.31E+04		31.573	
4	18	5.25E+04	4.80E+04	37.276	3.22E+01
	20	4.34E+04		27.115	
12	2	3.62E+04	3.58E+04	28.212	3.77E+01
	2	3.53E+04		47.262	
19	7	4.12E+04	4.31E+04	46.859	4.11E+01
	2	4.50E+04		35.317	
22	1	5.56E+04	5.34E+04	42.499	4.07E+01
	2	5.11E+04		38.974	
26A	7	3.62E+04	4.20E+04	37.074	3.28E+01
	9	4.77E+04		28.476	
26B	8	4.62E+04	3.84E+04	38.184	3.28E+01
	10	3.06E+04		27.400	
27	11	5.80E+04	5.93E+04	38.794	4.06E+01
	12	6.06E+04		42.499	
28	3	4.02E+04	3.97E+04	29.190	2.92E+01
	2	3.92E+04		29.168	
29	1	3.24E+04	2.92E+04	38.557	3.61E+04
	5	2.92E+04		N/A	
	7	2.64E+04		32.792	
	9	2.89E+04		36.899	

Table 13. Chloride ion permeability based on charge passed (AASHTO T277).

Charge Passed (Coulombs)	Chloride Permeability
>4000	High
2000-4000	Moderate
1000-2000	Low
100-1000	Very Low
<100	Negligible

Table 14. Field air permeability and laboratory rapid chloride ion permeability test results.

Site Number	Slab No.	Air Permeability (mL/min)			Rapid Chloride (Coulombs)		
		Data	Average of Site	Rating of Site	Data	Average of Site	Rating of Site
0	3	4.24	3.335	Good	1384	1434.5	Low
	4	2.43			1485		
1	16	19.52	16.465	Permeable	1982	1724.5	Low
	18	13.41			1467		
2	4	5	4.605	Good	1336	1283.5	Low
	3	4.21			1231		
3	1	7	6.58	Moderate	1400	1518.5	Low
	2	6.16			1637		
4A	3	17.45	18.59	Permeable	1850	1425	Low
	2	19.73			1000		
4	20	16.32	16.54	Permeable	2637	2641	Moderate
	18	16.76			2645		
12	2	12.37	12.94	Moderate	4775	3929.5	Moderate
	2	13.51			3084		
22	2	13.63	13.63	Moderate	1608	1254	Low
	1	13.63			900		
26A	9	13.63	13.63	Moderate	2076	1662.5	Low
	7	13.63			1249		
26B	10	13.64	13.64	Moderate	3341	2939	Moderate
	8	13.64			2537		
27	11	13.64	13.64	Moderate	2945	2073.5	Moderate
	12	13.64			1202		
28	2	19.233	14.76	Moderate	4125	2586	Moderate
	3	10.2833			1047		
29	1	N/A	16.42	Permeable	9133	7646.75	High
	5	10.84			4964		
	7	14.732			8011		
	9	23.69			8479		

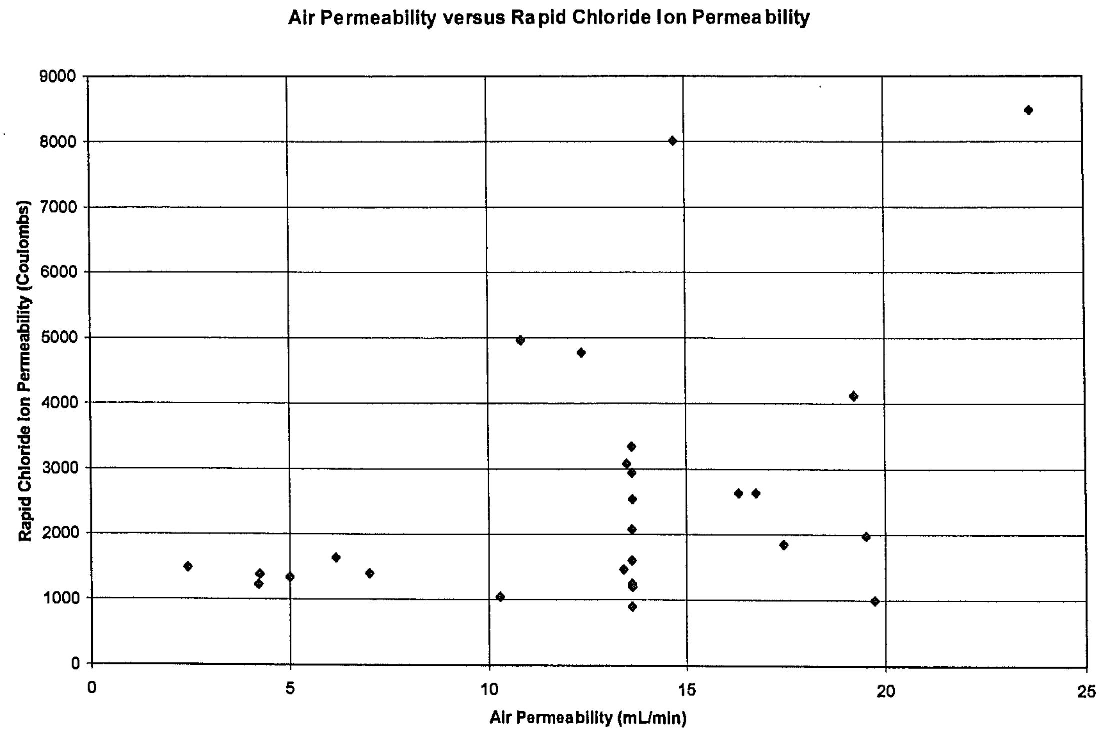


Figure 10. Plot of air permeability versus rapid chloride ion permeability.

One interesting observation is that although no readily observable trend exists between the aggregate type and the rapid chloride ion permeability test, this is not the case with the air permeability test. The three of the four highest air permeability measurements were made on concrete having slag coarse aggregate, including the non-distressed test site (Site No. 1). All of the slag sites tested (air permeability was not conducted on Site No. 19) had an average air permeability rating of "permeable" whereas only one of the sites constructed with natural aggregate (Site No. 3) received such a high rating. Similar trends were not observed with the rapid chloride ion permeability test.

MICROSTRUCTURAL ANALYSIS

The microstructural analysis of the concrete was conducted in accordance to the procedures presented in the FHWA guidelines (Van Dam et al. 2002a). As summarized in Chapter 2 of this final report, the pivotal stage in this evaluation is the use of the stereo OM to determine air-void system parameters and to make a general assessment of the condition of the concrete. Although diagnosis is possible through the use of the stereo OM alone, stains to accentuate sulfate minerals (barium chloride/potassium permanganate) and alkali-silica reaction product (sodium cobaltinitrite) were used in the evaluation. Also, thin sections were made and evaluated from specimens from all the test sites evaluated in this study.

Air-Void System Analysis (ASTM C457)

Slabs were cut from the core specimens and polished, and the stereo OM was used to conduct an ASTM C457 Procedure B – Modified Point Count Method. The summary of the results, including the computed original and existing air content and spacing factor, are provided in Table 15 and plotted in Figures 11 and 12. The dashed lines in Figure 12 show the range of normally accepted spacing factors. The raw data for the air void analysis can be found in Appendix C.

To assist in differentiating "infilled" air voids from hydrated cement paste, the slabs were stained with a solution of potassium permanganate and barium chloride. The chemicals stain sulfate-bearing minerals, such as ettringite, pink. Over time, such minerals commonly occur as secondary growths in the entrained air voids. Often, entrained air voids in deteriorated concrete are completely filled with secondary ettringite. The stain assists in the identification of filled air voids that may otherwise be mistaken for hardened cement paste. In the equations used to determine the air-void system parameters, filled air voids that are identified as paste would be considered to offer no protection to freeze-thaw damage. It is controversial whether filled air voids can protect the paste against freeze-thaw damage, and thus, air-void system parameters are computed for both the concrete in its original state, and the concrete in its existing state.

Table 15. Air-void system parameters determined using ASTM C457.

Site Number	Air Content (%)		Spacing Factor (mm)	
	Original	Existing	Original	Existing
0	7.5	7.3	0.121	0.129
1	9.3	8.6	0.161	0.188
2	6.2	6.1	0.180	0.203
3	6.6	6.4	0.234	0.257
4A	8.0	7.9	0.114	0.115
4	10.0	8.7	0.127	0.181
12	8.0	7.7	0.165	0.177
19	10.5	9.8	0.266	0.292
22	6.9	6.3	0.202	0.223
26A	6.2	4.8	0.204	0.276
26B	7.1	7.0	0.162	0.170
27	7.1	6.6	0.134	0.156
28	10.1	9.1	0.119	0.128
29	10.9	10.5	0.144	0.177

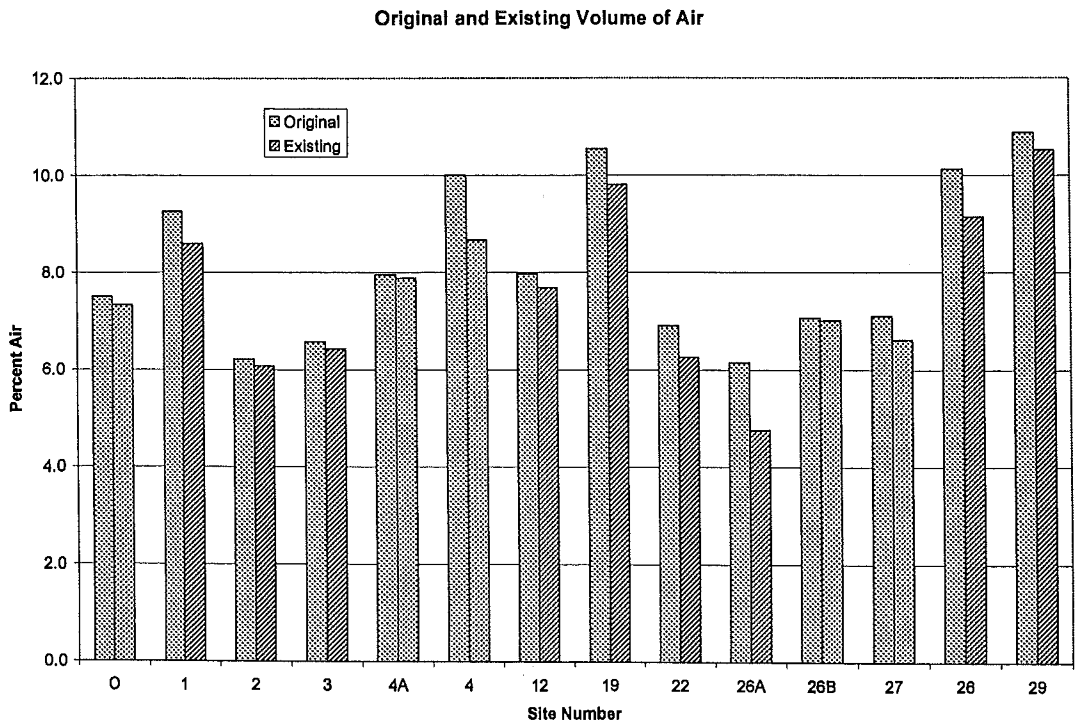


Figure 11. Original and existing air content from ASTM C457.

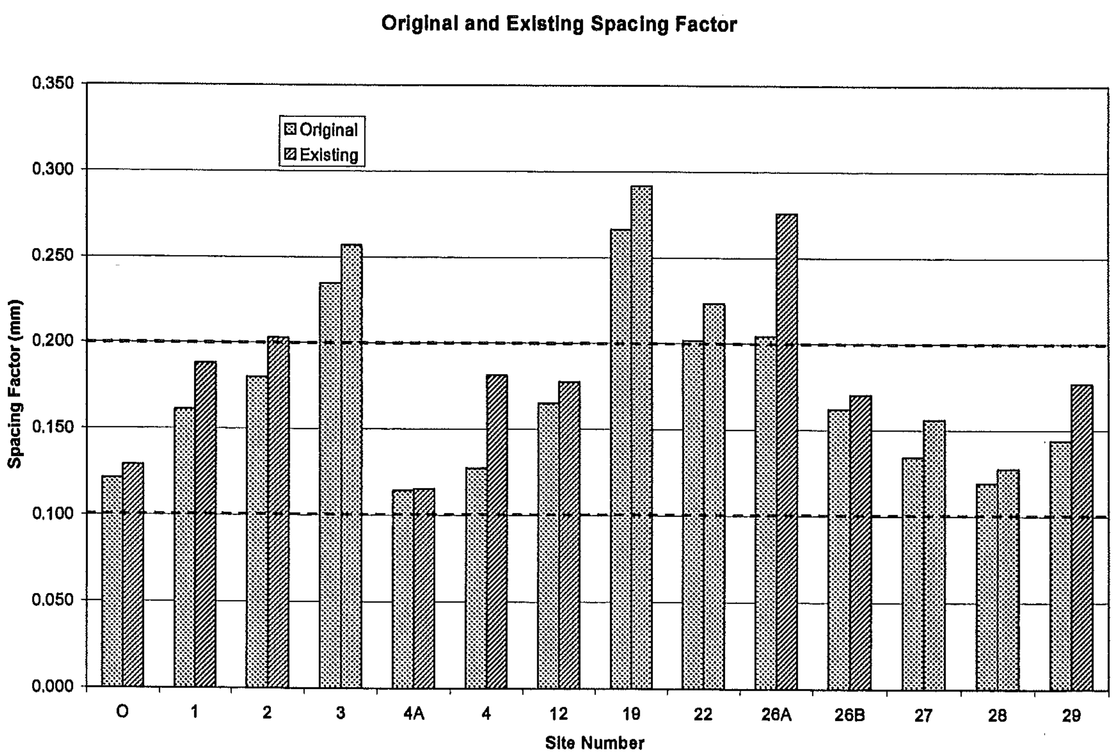


Figure 12. Original and existing spacing factor from ASTM C457.

Stereo OM and Staining

In addition to the slabs used for ASTM C457, the stereo OM provides a wealth of additional information. This includes general assessments regarding the fabric and quality of the concrete (i.e. segregation, bleed channels, etc.) to specific features indicative of concrete deterioration, such as alterations to the aggregate or paste. For example, aggregate freeze-thaw deterioration (D-cracking) is easily observed using the stereo OM, as is shown in Figure 13. In addition, polished slabs can be treated with solutions of sodium cobaltinitrite and potassium permanganate to stain alkali-silica reaction product (yellow) and sulfate minerals infilling voids and cracks (pink), respectively. Figure 14 shows an example of a sodium cobaltinitrite stained cracked reactive chert particle observed from concrete from Site No. 4A. As can be seen, the particle itself has turned yellow, as has the surrounding paste and cracks. This method is very useful in identifying alkali-silica reactive particles, and in determining to what extent the reaction product has migrated from the particle. Figure 15 shows a sample from the same site stained with potassium permanganate, which has stained sulfate minerals infilling the air voids a pink color.

Results from the staining test are qualitative in nature, and thus cannot be easily summarized in tabular format. Individual results and micrographs of this testing for each test site are presented in the site portfolios in Appendix D.

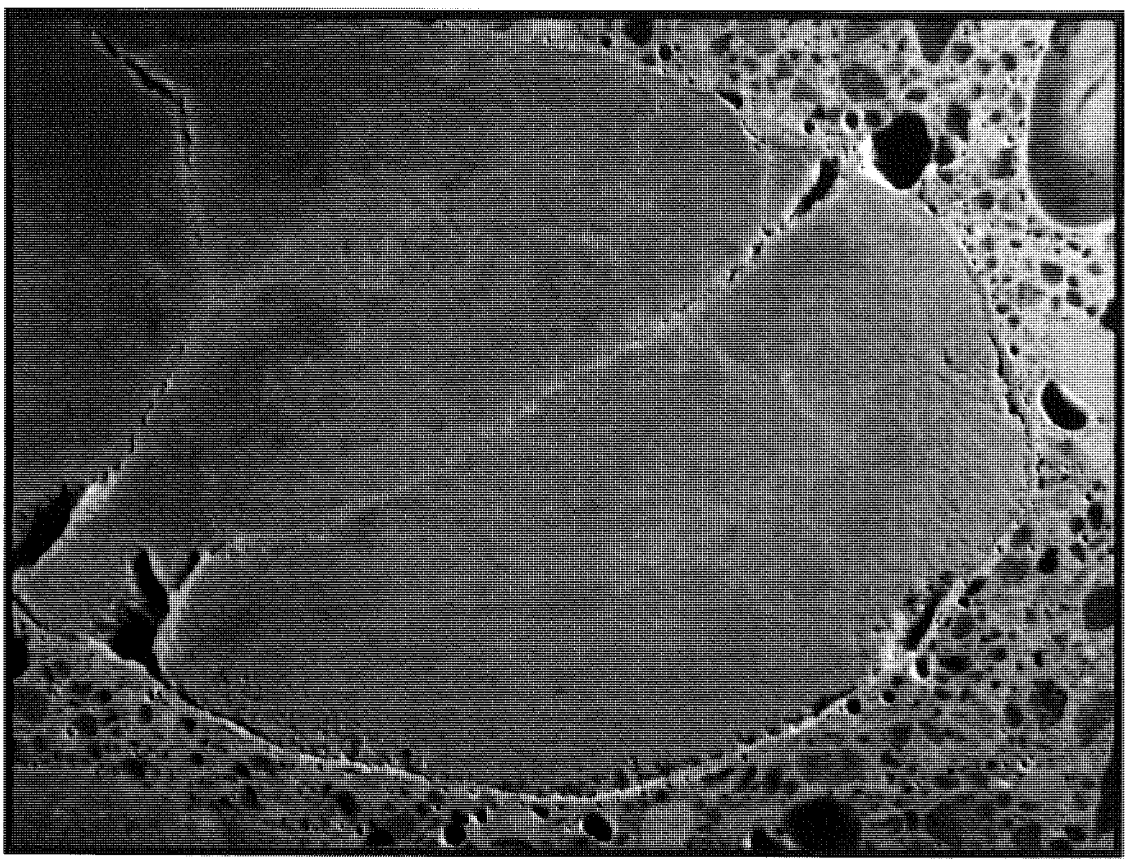


Figure 13. Fractured carbonate coarse aggregate from Site Number 22.

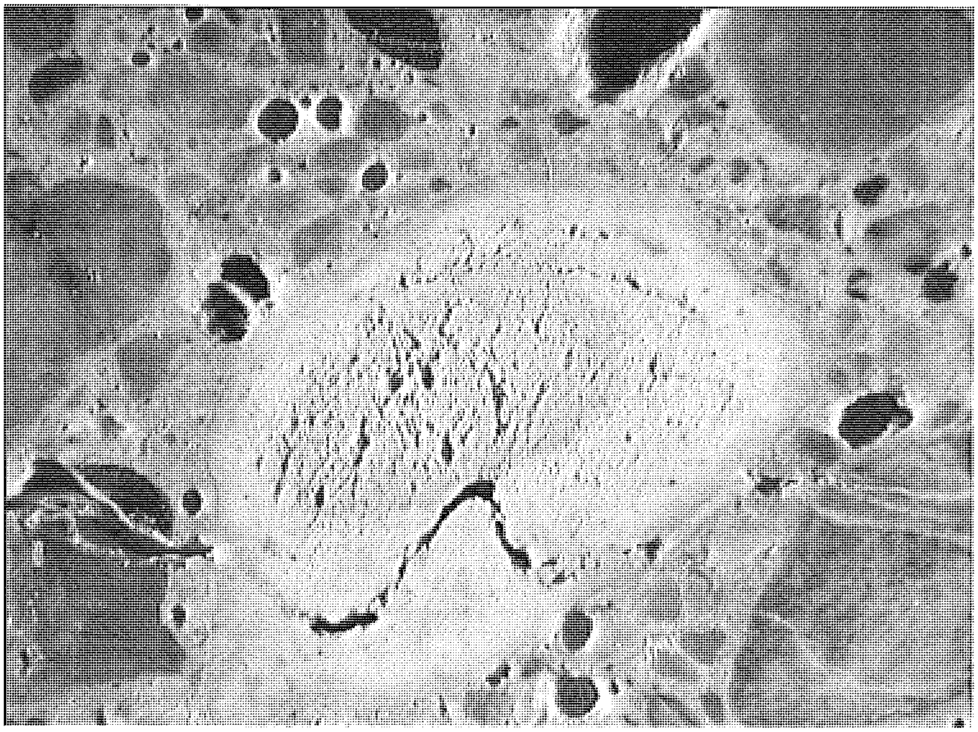


Figure 14. Cracked reactive chert particle, with cracks propagating through neighboring non-chert sand particles, magnified 15x.

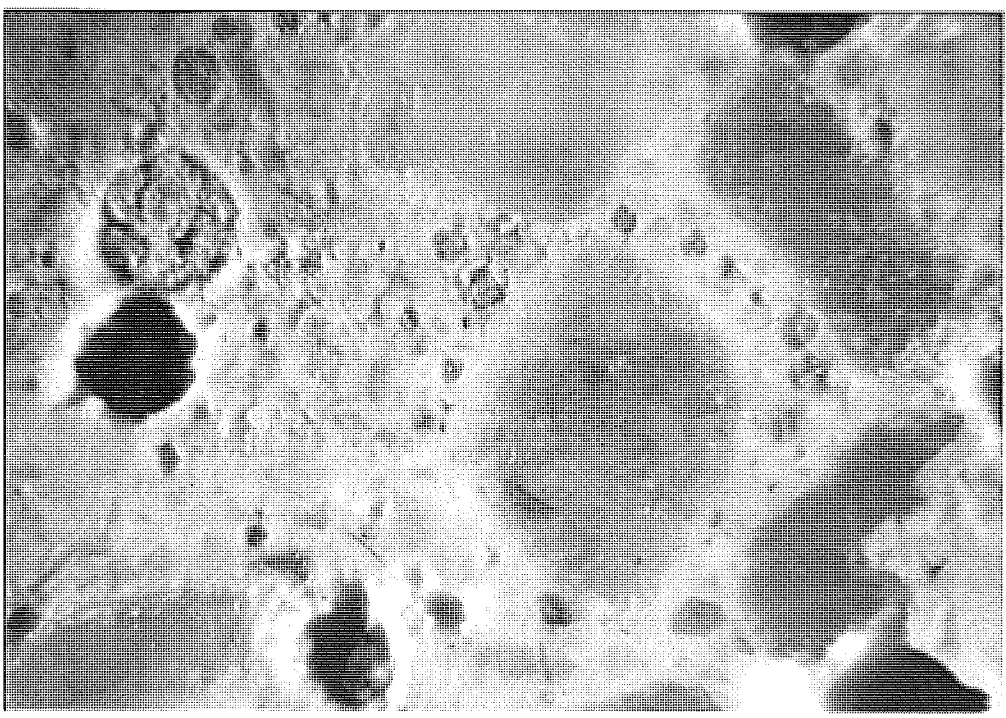


Figure 15. Microscope image of polished slab surface from Site Number 4 with ettringite filled air voids stained pink, magnified 83x.

Petrographic and SEM Examination

Fluorescent epoxy impregnated thin sections were made from specimens obtained from most of the test sites. An experienced petrographer viewing the thin sections in a petrographic and/or scanning electron microscope can identify various minerals and even make estimations of the *w/cm*. Figure 16 shows an example of a cracked reactive chert particle that is abundant in the cores obtained from Test Site No. 4. Figures 17 and 18 show a SEM data from Test Site No. 19. Petrographic and SEM data cannot be easily tabulated, and thus this information is included in the portfolios compiled in Appendix D.

Sodium and Potassium Determination

Given the prevalence of alkali silica reactive chert in some sections, an effort was made to determine the alkali content of the concrete. Since there were no records available for alkali content in the fly ash and cement, and no standard method available for determining available alkalis in hardened concrete, an extraction technique developed for the determination of exchangeable and soluble sodium and potassium in soils was adopted. The slabs from each core were crushed to a fine powder, and the alkalis were extracted with an ammonium acetate solution. The concentrations of potassium and sodium were determined by an inductively coupled plasma emission spectrophotometer.

The results from the test are expressed in weight percent elemental sodium and weight percent elemental potassium. In Table 16, the results were converted to the industry convention of kilograms of "Na_2O equivalent" per cubic meter of concrete, based on the total mass from the mix designs. This analysis was only run on cores from the Aggregate Test Site and those containing slag as a coarse aggregate.

The recalculation to kilograms of "Na_2O equivalent" per cubic meter of concrete allows comparisons to be made with industry standards for limiting total alkalis in the mixture. The Canadian standards specifying a maximum of 3.0 kg/m^3 Na_2O equivalent for mild protection, 2.2 kg/m^3 Na_2O equivalent for moderate protection, and 1.7 kg/m^3 Na_2O equivalent for strong protection. As can be seen in Table 16, two of the deteriorated test sites sections are above the recommended limits for "strong" protection whereas all three "fair" performing pavements are below the limit for "strong" protection.

Sulfate Determination

Secondary ettringite deposits in air voids are a common feature in many of the cores evaluated in this study. Previous research conducted at Michigan Tech has indicated that calcium sulfide present in blast furnace slag aggregate may dissolve and oxidize to become a source of sulfate and calcium in the concrete, perhaps encouraging the formation of ettringite in the entrained air voids (Peterson et al. 1999, Hammerling et al. 2000). A gravimetric procedure of sulfate determination was used according to British Standard 1881:Part 124. Methods for Analysis of Hardened Concrete: 1988. The weight percent SO_3 computed from the mix design assumes 3.5% by weight SO_3 for the cement and 4.5% by weight of the fly ash. Table 17 summarizes the results of this analysis.

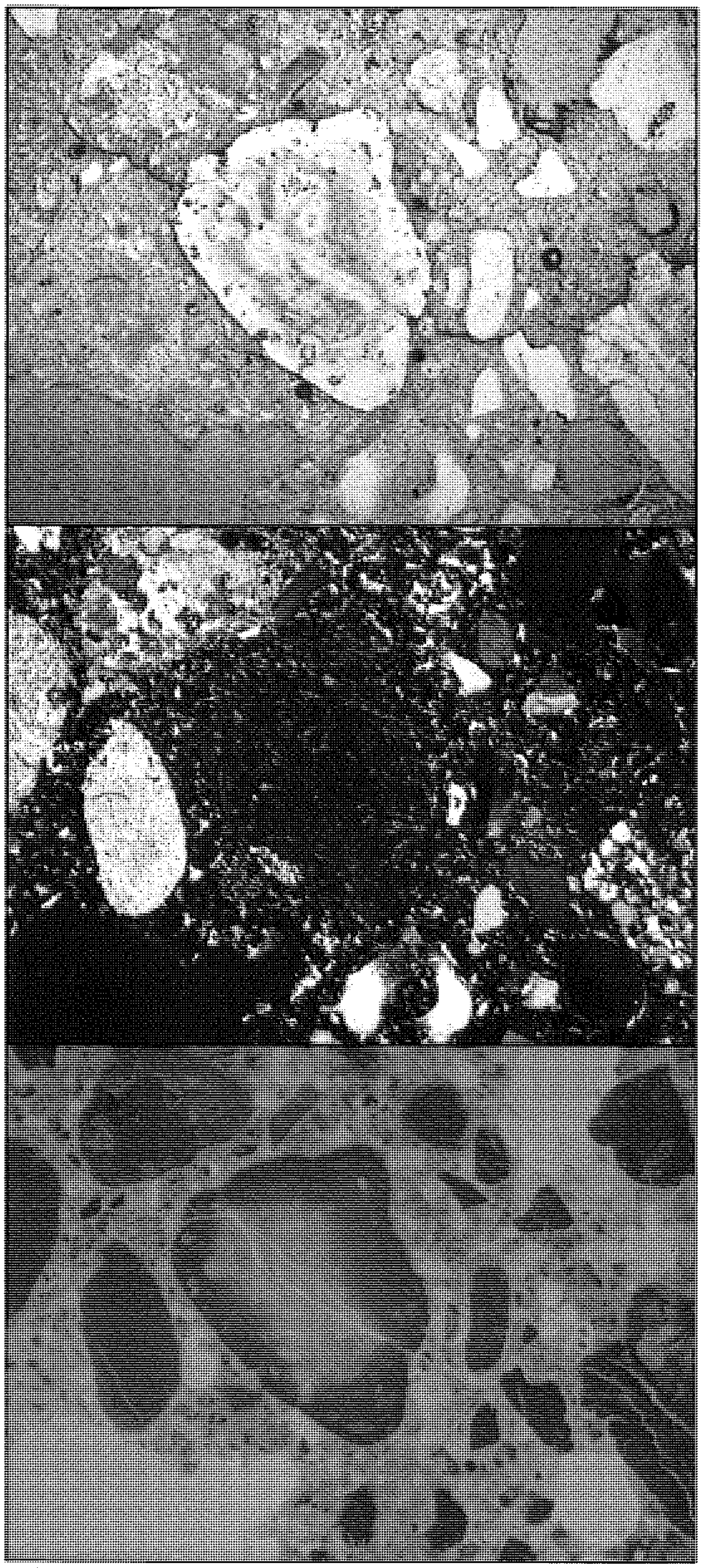


Figure 16. Example of cracked reactive chert from Test Site Number 4, magnified 40x. From top to bottom, plane polarized light, cross polarized light, and epifluorescent illumination.

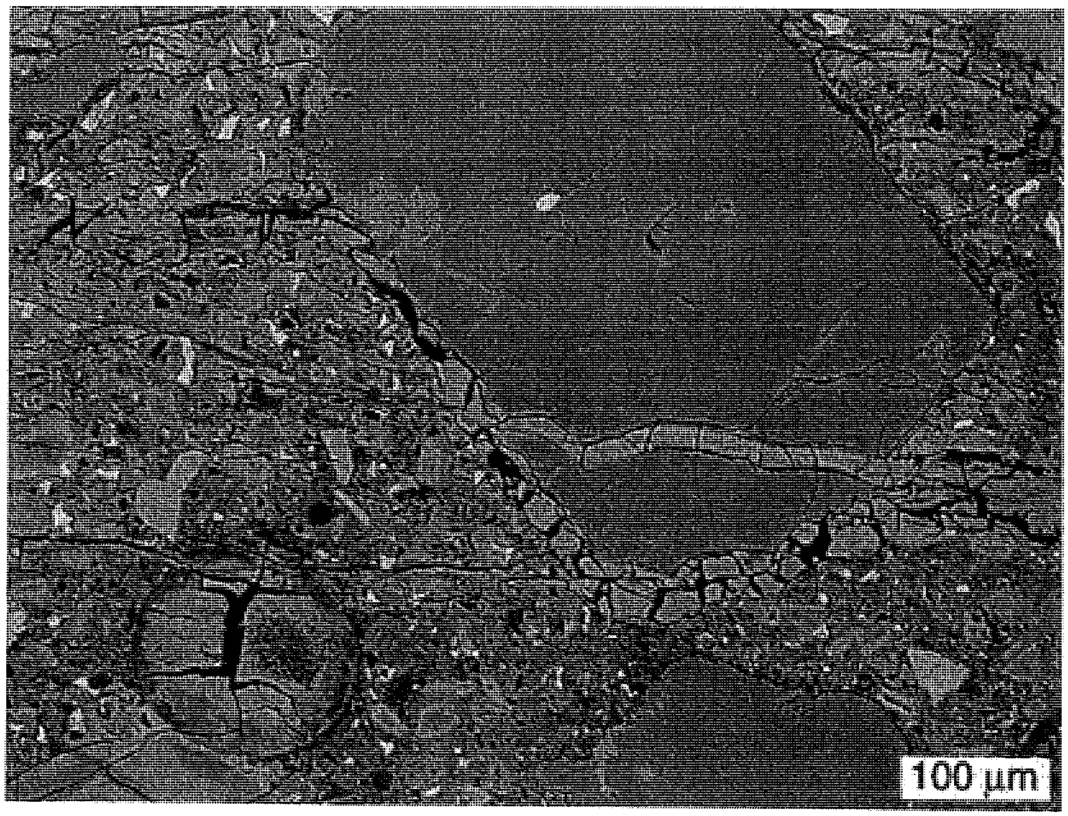


Figure 17. Back-scattered electron image of cracks and void filled with alkali-silica reaction product, Test Site No. 19.

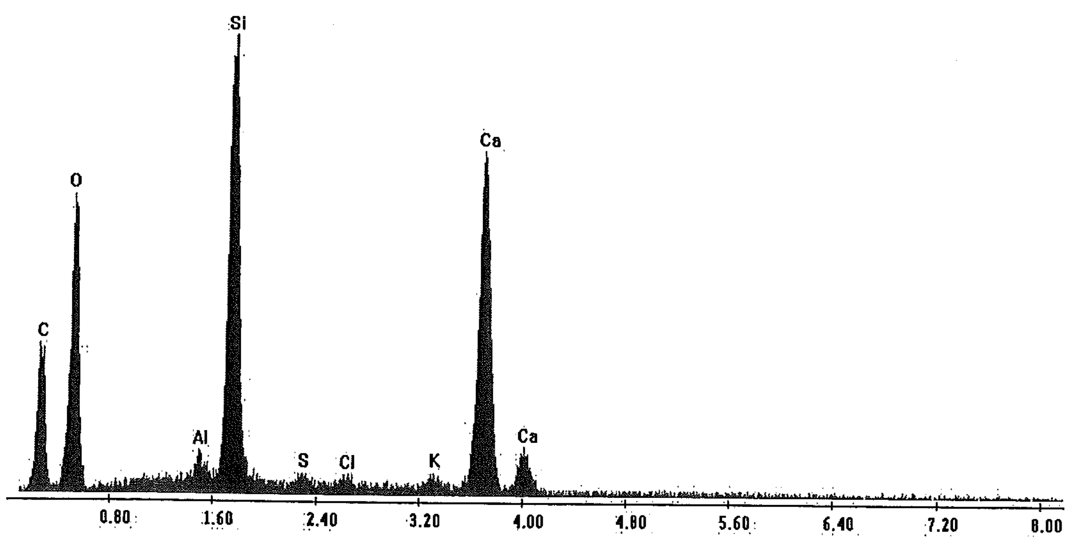


Figure 18. X-ray energy spectrum collected from alkali-silica reaction product shown in Figure 17.

Table 16. Summary of results of alkali extraction.

Test Site No.	Wt% Na	Wt% K	Na_2O Equivalent kg/m^3
0	0.021	0.036	1.29
1	0.026	0.038	1.46
2	0.021	0.038	1.35
3	0.021	0.033	1.27
4A	0.020	0.043	1.42
4	0.054	0.025	2.05
19	0.032	0.044	1.87
29	0.030	0.038	1.62

Table 17. Summary of results of sulfate extraction.

Test Site No.	Computed Wt% SO_3	Extracted Wt% SO_3
0	0.55	0.47
1	0.53	0.61
2	0.54	0.49
3	0.51	0.67
4A	0.54	0.48
4	0.53	0.67
19	0.48	0.68
29	0.46	0.57

SUMMARY OF LABORATORY TESTING

This chapter presented the data collected during the laboratory phase of testing. The data included compressive strength, rapid chloride permeability, air-void system analysis, stereo OM and staining, petrographic and SEM analysis, and chemical extractions. Detailed microstructural information cannot be easily tabulated and thus portfolios have been prepared and included in Appendix D. This information was used to interpret the cause of distress as presented in Chapter 5.

CHAPTER 5.0 — DATA ANALYSIS AND INTERPRETATION

It is difficult to easily present the data analysis and interpretation for a study such as this, since by its very nature there is no single thread connecting the various test sites under evaluation. To address this, this chapter is organized by drawing together the information and findings based on observed associations between the test sites. The following associations have been drawn:

- Group 1: The five test sites comprising the MDOT aggregate test site (Test Sites Nos. 0, 1, 2, 3, and 4A)
- Group 2: The three sites comprising the Clare Test Road (Test Site Nos. 26A, 26B, and 27).
- Group 3: The remaining three test sites that contain slag as a coarse aggregate (Test Site Nos. 4, 19, and 29).
- Group 4: The three remaining sites that have gravel as a coarse aggregate (Test Site Nos. 12, 22, and 28).

Detailed microstructural information is provided in Appendix D, which parallels this organization. This chapter will provided a summary of the data analysis and interpretation.

GROUP 1: SITES 0, 1, 2, 3, AND 4A – MDOT US-23 AGGREGATE TEST SITE

The portfolio for the test sites begins on page D-2 of Appendix D. The results of the evaluation of the cores received from the Group 1 test sites (Test Sites 0, 1, 2, 3, and 4A) are from pavements constructed in the year 1992. The cores, representing five different coarse aggregate types (two carbonate sources, two natural gravel sources, and one blast furnace slag source), are all from pavements in good condition. As would be assumed, differences in the microstructure exist in the concrete from the various test sites, primarily as a result of the coarse aggregate. These differences can be summarized as follows:

- In Test Site No. 3, the rims where the dolomite aggregates (Pit No. 58-08) contacted the paste stained positively for alkali-silica reaction product, a finding that is confirmed by petrographic analysis. Further, this reaction is associated with increased porosity and carbonation of the surrounding cement paste and densification of the aggregate rims. It seems that the darkened rims are due to an alkali-silica reaction where chert impurities in the dolomite have reacted with potassium from the cement paste. However, the darkened rims do not appear to be associated with any cracking or deterioration.
- It appears that the concentration of partially hydrated fly ash and cement particles is higher near the slag aggregate than in the bulk paste, with a corresponding increase in paste density. In addition, a green coloration is associated with the fly ash and the ferrite phase of the cement particles. It is suspected that the green color is due to the reduction of the iron in these materials, and is perhaps related to the calcium sulfide in the slag aggregate.

- The results of the sulfate extraction found that the measured sulfate content was lower than the predicted in 3 of the 4 mixtures containing natural aggregates. In the case of mixtures made with the slag (Pit No. 82-22) and the dolomite (Pit No. 58-08) coarse aggregates, the measured sulfate contents were higher than predicted. This finding, which was confirmed through repeating the test, suggests that slag and some natural aggregates may provide a source of internal sulfate.

In addition to differences observed between the mixtures due to the coarse aggregate type, the following observations were made:

- Two types of reactive fine aggregate particles were identified during the chemical staining procedures and petrographic analysis. One type is reactive chert that stained yellow after treatment with sodium cobaltinitrite and developed dense rims in contact with the cement paste. There was no cracking associated with the chert particles in any of the five of the test sites. The second reactive fine aggregate particle is a reactive siltstone that contains siliceous fossils that appear to be the reactive constituent. Typically, the siltstone particles are cracked, but the cracks do not extend into the surrounding cement paste. This is typical of the siltstone particles in all five of the pavements.
- The Class F fly ash, which is largely composed of highly reflective opaque minerals, is common to all five sites. It is interesting to note that in the presence of this fly ash, that no deleterious ASR was observed (no fracturing of aggregate particles) and that the higher than expected sulfate levels in two of the sites had no apparent ill effect. Also, the overall alkali levels in the mixtures were well below that needed to protect the concrete against ASR.

In total, the information provided through the analysis of the aggregate test site is quite interesting. The fact that alkali-silica reactivity was evident in two constituents of the fine aggregate is not surprising since this has been observed in other test sites as well. What is more important related to this observation is 1) at this time, the reactions are not causing damage (e.g. not cracking of aggregate or paste is observed), and 2) this provides a snap shot of the current condition that will provide a basis for comparison if the reaction becomes deleterious in the future. But the biggest question that begs to be asked is why in the case of these test sites is the reaction non-deleterious whereas in other test sites, the reactive chert particles have been strongly linked to deterioration. The answer might be most clearly linked to the relatively low total alkalinity of the mixtures, which never exceeded 1.7 kg/m^3, a value that the Canadian Standards (Fournier et al. 1999) deem as being "strong preventative action." It is also thought that the Class F fly ash may have had a mitigating effect.

Another interesting observation is the evidence that the dolomitic coarse aggregate (Pit No. 58-08) used in Test Site 3 not only contains alkali-silica reactive constituents but also may be a source of internal sulfates. This particular aggregate was also fairly porous. Again, no deleterious effects from either the alkali-silica reactive constituents was observed, possibly a result of the Class F ash and low alkalinity of the mixture. The Class F fly ash would also have been beneficial in mitigating any potential sulfate attack.

The final observation of interest is that Test Site No. 1, constructed with a slag coarse aggregate, had the same feature regarding reactivity in the fine aggregate as the other sites. Further, like Test Site No. 3, the extracted sulfate content was in excess of that predicted from the mixture design information, as was true of all the slag test sites studied. But again, no signs of deleterious reactions were observed.

GROUP 2: SITES 26A, 26B, AND 27, THE MDOT US10 CLARE TEST SITE

The portfolio for these Test Sites begins on page D-26 of Appendix D. The MDOT US10 Clare Test Site was developed to evaluate various pavement design elements, and as such, the concrete used was designed to be the same throughout. Although the three sites all have the same mix design, the base materials used at the sites were not the same. Both Test Sites Nos. 26A and 26B utilized an asphalt treated permeable base, while Test Site No. 27 was supported on an aggregate base. The pavements are all currently in relatively poor condition, suffering joint deterioration. The following summarizes the microstructural observations made for these three test sites:

- Although the concrete was supposed to be the same, there are some noticeable differences between that in Test Site 26A and the other two test sites. The first observation is that the hydrated cement paste in Test Site 26A appears to be more dense, indicating that the *w/cm* for this site may have been less than for the other two sites. In addition, the overall original air content at Test Site 26A was also less than that at the other two sites.
- One primary distress mechanism appears to be ASR associated with the various cherts, impure cherty carbonates, and sandstones coarse aggregate particles. These aggregate particles have cracked, and copious quantities of alkali-silica reaction product are observed filling these cracks, which extend into the paste.
- A potentially contributing factor is aggregate freeze-thaw distress, since many of the cracked coarse aggregates are fine-grained and porous, sharing similar properties to aggregates known to be susceptible to freeze-thaw deterioration. Thus, it is possible that the cracking in the coarse aggregate is a result of freeze-thaw deterioration, and that the alkali-silica reaction occurred once the concrete fabric was compromised.
- Infilling of the air-void system with ettringnite is common, but more so in Test Site No. 26A. In this case, a marginal air-void system is becoming less capable of providing protection against freeze-thaw damage. Such infilling is a common feature in distressed concrete, and it is unlikely that it is related to the initiation of the observed distress. Yet, it may contribute to the continuation of the distress once it has been initiated.

Based on these observations, a couple of things are evident. The first is that although efforts were made to keep the concrete the same over the entire Clare Test Road, it is apparent that some differences exist from test site to test site. This is not unexpected, but it must be recognized. In the three test sites examined, the hydrated cement paste in Test Site No. 26A clearly seems to be more dense, having lower capillary porosity, than that in Test Site Nos. 26B and 27. In addition, the results of ASTM C457 suggest that the air-void system in Site No. 26A may differ slightly from that in the other two test sites, and that more infilling of the air-void system with ettringite has occurred than in the other sites, possibly contributing to the progression of the distress.

The second point of interest is that the deterioration observed is almost certainly related to fracturing of the coarse aggregate particles. The concentration of the distress at the joints suggests that D-cracking (aggregate freeze-thaw deterioration) is likely to blame, but the microstructural analysis found a stronger link to ASR. This is not to say the aggregate freeze-thaw deterioration may not be contributing to the deterioration, but it is important to recognize that ASR is present in sufficient abundance that it must not be ignored. It is also worth noting that if a more durable source of coarse aggregate were used (both resistant to freeze-thaw damage and ASR), these problems would not exist.

GROUP 3: SITES 4, 19, AND 29: PAVEMENTS WITH BLAST FURNACE SLAG COARSE AGGREGATE

The portfolio for these Test Sites begins on page D-40 of Appendix D. The cores received from Test Site Nos. 4, 19, and 29 were from distressed pavements made with iron blast furnace slag coarse aggregate. They were constructed between 1976 and 1992, and are located in the Southeastern part of the Lower Peninsula. One of the test sites (Test Site No. 4) was ultimately included in a special study, the report of which is presented in Appendix E. The results of the microstructural evaluation can be summarized as follows:

- In all three pavement sections, reactive chert particles that are a constituent in the fine aggregate were aggressively and deleteriously alkali-silica reactive, leading to the formation of significant reaction product and associated cracking in the hydrated cement paste. This was true even though the extracted alkali contents of the two of the three sections would indicate "moderate" protection against ASR. The third section (Test Site No. 29) had an even lower extracted alkali content that would offer "strong" protection against ASR, but it was built in 1976 which may indicate that ultimately the deleterious reaction with the chert will occur given enough time. Only one section had fly ash (Test Site No. 4) and this was a Class C fly ash.
- In all cases, the extracted sulfate contents were in excess of that predicted from the mixture design. In addition, the degree of crack and air void infilling with ettringite was high. It was also observed that in many cases the calcium sulfide dendrites located near the aggregate-hydrated cement paste interface were "empty", indicating that potential dissolution of the calcium sulfide had occurred. This is one explanation for the origination of the excess sulfates measured through extractions, although further investigative work needs to be conducted.
- Often, in all three pavements, the cement paste in the immediate vicinity of slag coarse aggregate particles appears darker than the rest of the cement paste, primarily due to the presence of partially hydrated cement grains that are very abundant. In addition, calcium hydroxide crystals are also often found in the air voids near slag aggregate particles. Further, a green coloration is associated with the ferrite and aluminate phases of the cement grains near the interface. The implication of these findings is currently unknown, but without question some chemical interactions are

occurring between the slag coarse aggregate and the hydrated cement paste/pore solution in the vicinity of the paste-aggregate interface.

- At all three test sites, the measured original air content was significantly higher than that specified (10 percent or greater). Even though, one test site (Test Site No. 19) had an original spacing factor that would provide only marginal protection against paste freeze-thaw damage. Further, the infilling with ettringite has significantly altered the measured air-void system parameters, potentially reducing paste freeze-thaw durability.

In comparing the characteristics of these three sites to Test Site No. 1 (the slag coarse aggregate site in Aggregate Test Road) some notable similarities and differences are observed. Similarities include the densified hydrated paste zone with a concentration of unhydrated cement grains (and in some cases fly ash) in the vicinity of the slag coarse aggregate, the higher than expected extracted sulfate content, the green coloration associated with the ferrite and aluminate, the high concentration of calcium hydroxide, and relatively high original air contents. Regarding the last point, it is possible that the high air contents simply reflect the difficulties encountered in differentiating round entrained air bubbles in gray hydrated cement paste from round bubbles in gray slag coarse aggregate. But, it also may be a result of the difficulty in controlling entrained air in slag concrete.

The observation of the densified paste and unhydrated cement grains adjacent to slag particles might be due to a decrease in the *w/cm* at the aggregate-paste interface. This is opposite of what commonly occurs with natural aggregates, where the interface is characterize by locally high *w/cm* due to water on the aggregate surface. It is possible in the case of slag, mixing water continues to be absorbed into the slag particle, thus lowering the water at the particle surface.

The most notable difference between the sites is performance, whereas Test Site No. 1 is performing well, both macroscopically and microscopically, whereas the other sites are performing poorly. It is possible that this may change with time since Test Site No. 1 is fairly new (1992), but it was constructed in the same year as Test Site No. 4, so age alone cannot explain the difference. In all the deteriorated test sites, the chert constituent in the fine aggregate is reacting with vigor, producing significant quantities of alkali-silica reaction product and associated cracking. In contrast, this same chert constituent in Test Site No. 1 shows signs of being reactive, but is not deleterious at this time. It is possible that the total alkali contents, based on the results of the extractions, can be partially responsible for this observation since they are very low in Test Site No. 1 (1.46 kg/m^3) versus Test Site No. 4 (2.05 kg/m^3), but this difference alone is so small that it is unlikely to be the primary reason for the difference[1]. It is more likely that the Class F fly ash used in Test Site No. 1 is the reason for its enhance durability since it is well known that Class F fly ashes are far more effective in mitigating ASR and sulfate attack than Class C fly ashes as is discussed in the next chapter.

[1] Note that the Special Report on US-23 finds a stronger correlation between the extracted alkali content and the occurrence of distress.

GROUP 4: SITES 12, 22, AND 28, PAVEMENTS WITH NATURAL GRAVEL COARSE AGGREGATE

The portfolio for these Test Sites begins on page D-60 of Appendix D. Test Site Nos. 12, 22, and 28 were from distressed pavements made with natural gravel coarse aggregate. Two of these test sites were built in 1957, and the third in 1960, so they are over 40 years old, and were still in service at the time of inspection. It seems likely that these pavements would have continued in service for many more years if it were not for the MRD. In general, compared to the typical concrete mixture used today, the concrete mixtures have a low cement content and large maximum aggregate size. The results of the microstructural evaluation can be summarized as follows:

- Unlike the concrete in the test sites containing slag coarse aggregate, the fine aggregate chert constituents, although showing signs of reactivity, have not reacted in a deleterious fashion. There is no cracking associated with these fine chert particles.
- On the other hand, coarse aggregate chert particles present in the natural gravel used in Test Site Nos. 12 and 28 do show signs of being deleteriously reactive, being linked both to alkali-silica reaction product and cracking. These same aggregates have properties that would make them potentially susceptible to aggregate freeze-thaw deterioration.
- The large coarse aggregate carbonates particles are porous and show significant cracking in the vicinity of joints. These aggregate have the porosity characteristic of those susceptible to D-cracking, having all the diagnostic features of suffering aggregate freeze-thaw deterioration.

It seems clear that aggregate freeze-thaw deterioration is the major contributor to the distress observed. The large, porous carbonate aggregates are highly susceptible to this distress mechanism. To a lesser degree, so are the chert coarse aggregate particles in two of the three sites, and they are also susceptible to ASR. The use of MTM 115 has prevented the use of such poor quality aggregates in newly constructed pavements, and it is a testament to the effectiveness of this test method that no recently constructed pavements has signs of aggregate freeze-thaw deterioration.

SUMMARY

Table 18 presents a brief summary of the distress mechanism identified through this laboratory study in each of the test sites. In total, four potential distress mechanisms (paste freeze-thaw deterioration, aggregate freeze-thaw deterioration, alkali-silica reactivity, and sulfate attack) were identified as being potential contributors to the observed distress. In the table, a "P" is used to designate what is considered to be the primary distress mechanism and an "S" a secondary mechanism. A "?" designates a mechanism that may or may not be contributing to future distress in the Aggregate Test Site, as conditions exist that make it possible. Future monitoring of these test sites will help establish whether deleterious reactions will occur. An "M" indicates that a marginal air-void system is present.

Table 18. Summary of distress mechanisms identified in this study.

Test Site No.	Distress Mechanism			
	Paste F-T	Aggregate F-T	ASR	Sulfate Attack
0[1]			?	
1[1]			?	?
2[1]			?	
3[1]	M		?	?
4A[1]			?	
4			P	S
12		P	S	
19	M		P	S
22	M	P		
26A	M	S	P	
26B		S	P	
27		S	P	
28		P	S	
29			P	S

It is also clear that this is not a full inventory of all MRDs that might be at work in Michigan's pavements. For example, there are many incidences of corrosion of reinforcing steel and dowel bars. These were not included in the study since their presence, cause, treatment, and prevention are well understood. It is also likely that other MRDs were "missed" since a fairly small group of pavements was studied. But overall, it is believed that this study has captured a good representation of the MRDs that are currently present.

[1] Test Site Nos. 0 through 4A are from the non-distressed Aggregate Test Site.

CHAPTER 6.0 — TREATMENT AND PREVENTION OF OBSERVED MRD

This chapter presents recommendations for treatment and prevention of the MRDs observed in the course of this study as summarized in Table 18. The treatment recommendations are general in nature since a detailed project evaluation would need to be conducted on each test site to more fully characterize the severity and extent of distress before specific recommendations can be made. The prevention recommendations are directed at addressing the specific MRDs observed, and do not consider site-specific constraints, such as material availability.

As discussed in Chapter 5 of this report, the potential MRD mechanisms identified in the test sites include (in the order listed in Table 1):

- Paste freeze-thaw deterioration (Test Site Nos. 3, 19, 22, and 26A)
- Aggregate freeze-thaw deterioration (Test Site Nos. 12, 22, 26A, 26B, 27, and 28)
- Alkali-silica reactivity (Test Site Nos. 4, 12, 19, 26A, 26B, 27, 28, and 29)
- Internal sulfate attack (Test Sites Nos. 4, 19, and 29)

The following discussion, which is largely based on the FHWA guidelines (Van Dam et al. 2002a), addresses the treatment and prevention strategies for each deterioration mechanism individually.

TREATMENT RECOMMENDATIONS

This section presents information on selecting the most appropriate treatment or rehabilitation options to address MRD in concrete pavements. In addition, means of evaluating and selecting the proper repair materials and techniques are also presented. 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 19 presents a flowchart for selecting the preferred treatment or rehabilitation option (Van Dam et al. 2002a). Although the overall process is the same regardless of the type of MRD exhibited, treatments and rehabilitation strategies are further broken out by MRD type. The overall process also considers the means to address pavements exhibiting multiple distress types, including distresses that are not caused by durability problems.

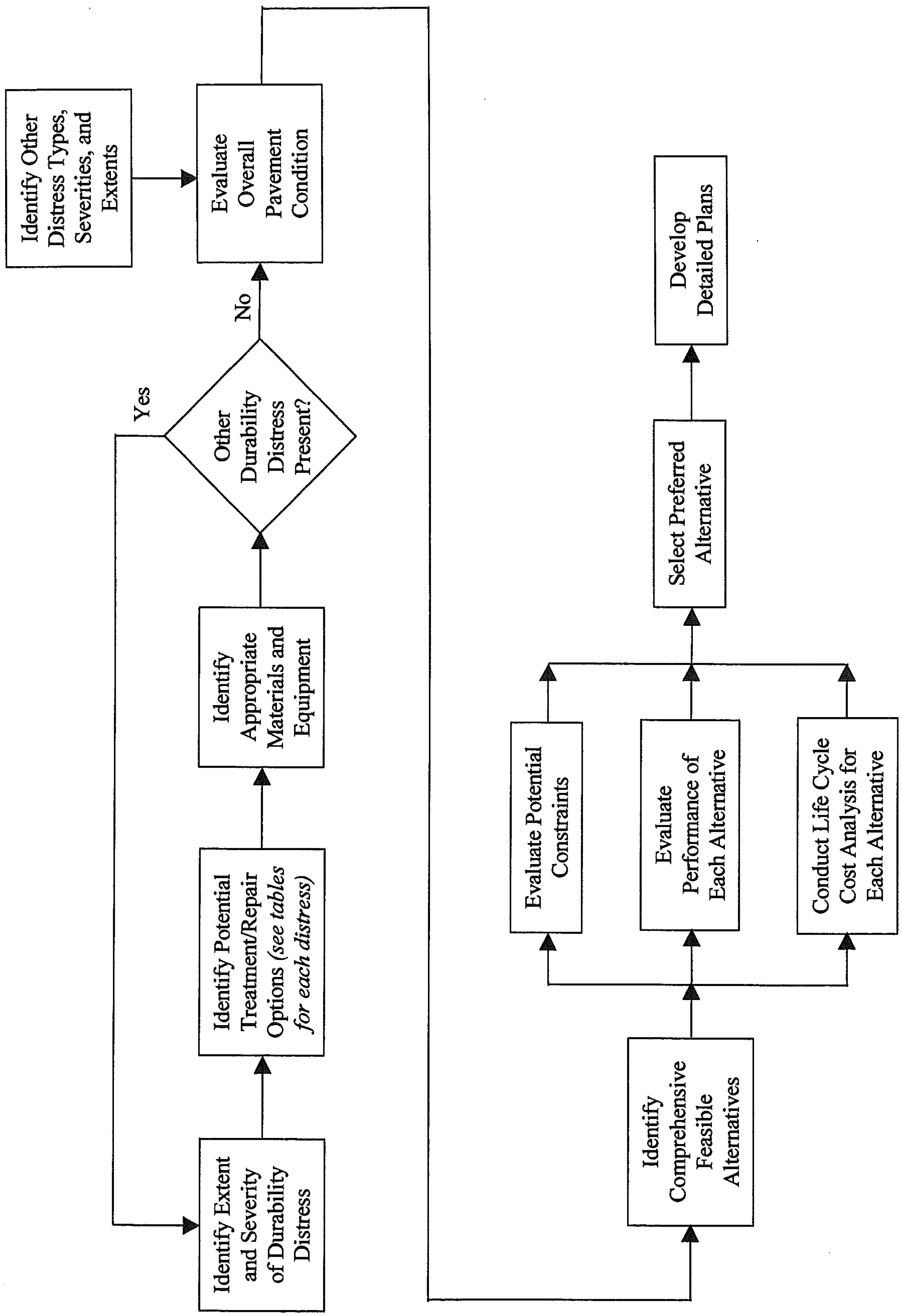


Figure 19. Flowchart for selecting preferred treatment and rehabilitation options (Van Dam et al. 20022a).

The first step is to identify the extent and severity of MRD in the existing pavement. 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.

Techniques and Materials for Treatment and Rehabilitation

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). For the sake of avoiding redundancy, a brief overview of all potential treatment and rehabilitation alternatives for addressing MRD is first presented. The practical alternatives for each MRD are then presented later under separate headings.

Available treatment methods include:

- Chemical treatments
- Joint and crack sealing
- Crack filling
- surface sealing
- Retrofitted drainage

The purpose of the various treatments is to arrest the mechanisms that cause distress or at least delay the progression of the distress mechanism. The different treatment methods work in a variety of different ways. For example, chemical treatments affect the chemical reactions that cause the distress. Retrofitted drainage, on the other hand, can be effective if it limits the amount of available water in the pavement system that would otherwise exacerbate the distress.

Rehabilitation 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, reconstruction, and recycling.

The most economical solution to MRD is to address the problem in the design and construction phase of the project and thereby prevent the distress from ever occurring. This is discussed in the next part of this chapter. However, durability problems do occur, as evidenced by the many miles of existing pavement exhibiting MRD. Therefore, there remains a primary need to apply methods that can address durability problems that exist in the field.

Overview of Treatment Methods

Chemical Treatments

Chemical treatments are more often used in fresh concrete to prevent deleterious reactions. In some cases, they can also be effective if applied to the surface of existing concrete pavements. The main shortcoming in this approach is obtaining significant penetration of the chemical treatment through the depth of the slab. Several applications of the treatment can help increase the depth of penetration.

Chemical treatments can work in several ways, as follows:

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

Treatment with lithium compounds is an example of a chemical treatment that has potential to arrests an existing alkali-silica reaction. Other types of treatments are discussed as appropriate for each distress type.

Joint and Crack Sealing

Moisture is a key ingredient to the development of many MRDs. As a result, these distress types are often observed to be much more severe at joints and cracks, where moisture can penetrate the concrete from several surfaces. Sealing joints and cracks can help reduce 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 (e.g. water beneath the slab in the base/subgrade can be substantial) and what's the volume of this water?
- How long will the sealant be effective at reducing the infiltration of water?

It is impossible to completely eliminate the intrusion of moisture at joints and cracks, especially for an extended period of time. 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). This confirms that moisture can continue to penetrate the concrete at joints and at other locations, making this approach a short-term solution under the best of circumstances.

Crack Filling

Crack filling must first be disassociated from crack sealing. In the context of concrete pavements, crack filling refers to the filling of surface cracks (e.g., map cracking) with a material that penetrates into cracks, not the sealing of individual full-depth transverse and longitudinal cracks. The primary purpose of crack filling is to strengthen the concrete pavement. Crack fillers penetrate into surface cracks and effectively "glue" the concrete pieces together. However, crack filling provides the added benefit of partially sealing the surface to prevent infiltration of moisture and other harmful constituents. An example of a crack filling material used by highway agencies on MRD pavements is high molecular weight methacrylate (HMWM).

Surface Sealing

Surface sealing or coating helps prevent the ingress of moisture into the pavement, which can help deter the initiation or alleviate the extent of moisture-induced distress. However, sealers only treat the surface, so concrete pavements are still exposed to moisture from the sides and bottom. Consequently, their effectiveness is limited, especially on slab-on-grade designs in which the slab is in direct contact with the subgrade. 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 to line them with a water-repellent substance (Campbell-Allen and Roper 1991). Penetrants can be considered a misnomer, because they do not really penetrate very far beneath the surface. Like all sealers, they are thus subject to being worn off by traffic. 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. Consequently, sealers are most effective at limiting constituents that infiltrate from the pavement surface, such as water and chloride ions from deicing salts.

Retrofitted Drainage

The addition of retrofitted drains will, in theory, remove drainable moisture from under the slab and at joints and cracks, which would assist in slowing or delaying MRD, since most deleterious expansion involves water (e.g., expansion due to ice formation, ASR gels imbibing water, etc.). 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.

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 the drying effects of the environment (such as the sun and wind), even a light rain can saturate the underlying layers, which may 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. 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.

Overview of Rehabilitation Methods

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.

Partial-depth repairs are not an ideal repair for many MRDs because the deterioration is often worse at the bottom of the slab, where moisture and deleterious chemicals are more readily available. In such cases, partial-depth repairs are not a practical treatment because they do not address the root cause of the deterioration, and the patch itself will likely become debonded and quickly deteriorate.

Full-Depth Repairs

Full-depth repairs are generally a better alternative than partial-depth repairs for addressing pavement deterioration caused by MRD concentrated at joints or cracks. 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, most types of MRD are generally more severe along joints and cracks due to increased exposure to water and deleterious substances, which makes full-depth repairs an appropriate repair method.

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.

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. This is particularly true if there are isolated instances where distress is not confined to the joint area. The problem is that MRD generally occurs throughout the entire project and is not likely to be limited to an isolated number of slabs. Thus, it should be recognized that slab replacement does not completely address the durability problem (unless all slabs are replaced). Slab replacement can 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 the concrete pavement surface. This corrects surface irregularities, provides a smooth riding surface, and improves the frictional characteristics. For most durability problems, the purpose of diamond grinding is simply to "buy some time" until more permanent rehabilitation can be conducted. Scaling of the concrete surface caused by deicing salts is an example of a condition that can be corrected by diamond grinding. Diamond grinding is also an effective method in conjunction with partial-depth and full-depth repairs to restore the ride quality. As with other rehabilitation methods, diamond grinding does not directly address the cause of the durability problem unless the problem was isolated to the pavement surface as can occur when poor finishing practices have contribute to the distress.

Overlay

The feasibility of an overlay for addressing MRD depends largely on the type and extent of MRD. An AC overlay or unbonded PCC overlay can provide improved service and extended life if used under the right circumstances. An AC overlay requires that the existing concrete pavement not exhibit substantial areas of deterioration that will quickly reflect through the overlay; otherwise, extensive preoverlay repairs are required. Unbonded PCC overlays are more forgiving, but are also more expensive and create additional concerns such as grade changes and overhead clearances. Bonded PCC overlays are not recommended for concrete pavements exhibiting MRD. It has also been observed in some cases that MRD in an existing pavement has been accelerated after application of an overlay.

Pavement Reconstruction

The most extreme alternative is total reconstruction of the pavement. This solution corrects an MRD problem and will prevent its recurrence if deleterious materials are not used again or are accounted for in the mix design. Because durability problems are not often limited to isolated areas within the pavement, reconstruction is often the only practical alternative, especially where the MRD is extensive and has progressed to high-severity levels. In these cases, the guidance provided in the latter portion of this guideline should be used to ensure that a durable pavement is constructed.

Pavement Recycling

Another form of reconstruction is pavement recycling, which involves removal and crushing of the existing concrete for use as aggregate in the reconstructed pavement. Recycling offers several benefits over reconstruction with all new aggregate, 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 conventional mix design procedures (e.g., crushing coarse aggregate to smaller size to prevent freeze-thaw deterioration of aggregate, addition of pozzolans to mitigate ASR, etc.) are required to prevent or limit the recurrence of durability problems in the recycled pavement. With these adjustments, however, recycling of concrete pavements with MRD can provide performance comparable to that of conventional mixes (Wade et al. 1997). In some cases, such as alkali-carbonate reactivity, recycling is not an option as this is the one aggregate related MRD that cannot be effectively mitigated.

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. The following sections provide guidance, broken out for each particular MRD type observed in the course of this project as affecting concrete pavements in Michigan, for selecting feasible alternatives. 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 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.

<u>Freeze-Thaw Deterioration of Cement Paste</u>

Freeze-thaw deterioration of cement paste is caused by repeated freeze-thaw cycles of the saturated cement paste. Freezing of moisture within the concrete produces internal stresses that lead to deterioration if an adequate air-void system does not exist.

Treatment methods focus on eliminating, or at least reducing, available moisture or freezing temperatures. Similarly, rehabilitation methods to address freeze-thaw deterioration of cement paste are considered temporary fixes to restore serviceability and extend the life of the pavement until more permanent rehabilitation techniques are conducted.

Available Treatment Methods for Freeze-Thaw Deterioration of Cement Paste

Treatment methods to address freeze-thaw deterioration of cement paste focus on reducing the amount of moisture or the number of freeze-thaw cycles. If neither moisture nor freeze-thaw cycles are present, the deterioration process will not take place. However, moisture is available from a variety of sources, so eliminating moisture in the pavement is difficult, if not impossible. Likewise, freezing temperatures are present throughout the entire State of Michigan. Feasible treatment methods include sealing joints and cracks and applying a surface seal. To be effective, these measures need to be performed during the early stages of the deterioration process.

Sealing joints and cracks is one means of reducing the amount of infiltrated water. Although this method will never be totally effective, it can reduce the amount of excess moisture that is allowed to infiltrate into the pavement system. The effect (if any) that this reduction in moisture infiltration will have on reducing freeze-thaw deterioration is debatable.

Another option is the application of a surface sealer. This treatment method forms a penetration barrier on the pavement surface, which expels moisture much like wax on a car. Freeze-thaw deterioration of cement paste is most extensive along the concrete surface, such as the top and bottom of the slab, where the concrete is subjected to greater exposure to the environment. Surface sealers can be effective for addressing freeze-thaw deterioration at the pavement surface, and may be very effective if the air-void system is compromised only at the pavement surface due to poor finishing during construction.

Available Rehabilitation Methods for Freeze-Thaw Deterioration of Cement Paste

The purpose of rehabilitation methods for addressing freeze-thaw deterioration of the cement paste is to restore the serviceability and extend the life of the pavement. Because of the difficulties in eliminating either moisture or freezing temperatures, rehabilitation methods are likely the best alternative on pavements in which the overall air-void system is inadequate. Feasible rehabilitation methods include partial-depth repairs, full-depth repairs, diamond grinding, overlays, reconstruction, and recycling.

Partial-depth and full-depth repairs are candidates where freeze-thaw deterioration is confined to joints and cracks, or small localized areas. The determining factor between the two approaches is the depth of deterioration. For partial-depth patching to be effective, the deterioration needs to be limited to the upper one-third of the slab. Cores taken at representative areas of freeze-thaw deterioration can help determine the depth of deterioration and the potential for future deterioration. If only isolated slabs are affected by surface scaling due to a poor air-void system at the surface, partial-depth inlays may be effectively used. When damage is found during construction to be more extensive than anticipated, the repair area should be made wider and converted to a full-depth patch, if necessary. The repair will not perform adequately if the deterioration is not completely removed.

If the freeze-thaw deterioration is more extensive, diamond grinding may be a more cost-effective alternative. To be feasible, the deterioration must be limited to the pavement surface. Scaling and map cracking isolated to the surface are examples of distress types that are good candidates for diamond grinding. Although diamond grinding will remove deteriorated areas from the pavement surface, it must be recognized that a new layer of concrete will be exposed and will also deteriorate with time if the air-void system is not adequate. Diamond grinding is also an effective method to restore ride quality in conjunction with other repair methods such as partial-depth and full-depth repairs.

Another rehabilitation option is to construct an overlay. Either an AC overlay or an unbonded PCC overlay is feasible. 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. 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 (Janssen and Snyder 1994).

In general, bonded PCC overlays are not recommended over pavements with durability problems because of the strong likelihood of the layers becoming debonded at the interface if the deterioration continues. On the other hand, in the special case where poor finishing techniques are solely responsible for the deterioration (e.g., over-finishing negatively affected the air-void system, water was added during finishing, bleed water was trapped, etc.), the complete removal of the susceptible layer and replacement with a bonded overlay may be a viable option.

Selection Guidelines for Freeze-Thaw Deterioration of Cement Paste

A summary of the feasible alternatives for pavements exhibiting freeze-thaw deterioration of cement paste is presented in Table 19. The selection of feasible alternatives depends on the extent and severity of distress observed during the field data collection process. As described in Table 1, paste freeze-thaw deterioration is most often manifest as map cracking, scaling, and/or spalling. The severity of the distress can be assessed using the following guidance:

- Low severity is characterized by tight cracks and very little scaling of a depth less than 3 mm.
- Medium severity is characterized by moderate scaling having a depth of 3 mm to 12 mm leaving coarse aggregates exposed. Delamination is usually evident upon close examination.

Table 19. Selection of feasible alternatives to address freeze-thaw deterioration of cement paste (Van Dam et al. 2002a).

Severity	Extent	Feasible Alternatives	Comments
Low	Corners	Seal the pavement Seal joints and cracks	Treatments should be aimed at limiting the amount of available moisture and delaying the progression to higher severity levels.
	Transverse and longitudinal joints	Seal the pavement Seal joints and cracks	
	Entire slab	Seal the pavement	
Moderate	Corners	Seal joints and cracks Full-depth repairs	Limit moisture to prevent further deterioration.
	Joints and cracks	Seal joints and cracks Partial-depth repairs Full-depth repairs Diamond grinding	
	Isolated areas	Partial-depth repairs Full-depth repairs Inlay[1] Slab replacement	Scaling of surface requires repair; cannot be treated.
	Entire slab	Diamond grinding Overlay	Damage is too extensive to repair each area.
High	Corners	Partial-depth repairs[1] Full-depth repairs	Deterioration is too severe for treatment; deteriorated areas must be removed and replaced.
	Joints and cracks	Partial-depth repairs[1] Full-depth repairs Diamond grinding Recycling Reconstruction	
	Isolated areas	Partial-depth repairs[1] Full-depth repairs Inlay[1] Slab replacement	Scaling of surface requires repair; cannot be treated.
	Entire slab	Diamond grinding Overlay Recycling Reconstruction	Deterioration is too severe and too extensive for treatment or restoration.

[1] Appropriate if laboratory analysis confirms that poor air-void system is isolated at the surface due to poor finishing during construction.

- High severity is characterized by extensive scaling having a depth in excess of 12 mm. Coarse aggregate are exposed and in some cases missing. Patching may be evident in affected area. Delamination is usually evident upon close examination.

The final selection of the preferred alternative will depend on other factors as well, such as the cost of the alternative and its compatibility with the overall plans for the pavement section.

The severity of deterioration largely determines whether the most appropriate alternative will be a treatment or rehabilitation method. As with any distress, treatment methods are more effective for addressing distress in the early stages of deterioration (generally, low-severity distress). These methods are designed to prevent or arrest the development of freeze-thaw deterioration. Conversely, rehabilitation methods are more effective for addressing distress in the later stages of deterioration. These methods concede to the deterioration mechanism by removing and replacing the distressed area.

The extent of deterioration is also a controlling factor in the selection process. Freeze-thaw deterioration of cement paste can be worse at the bottom of the slab, which is exposed to moisture trapped in the underlying layers, or at the slab surface, which is exposed to harsher freeze-thaw conditions. In addition, poor finishing may compromise the air-void system only at the surface, whereas it may be adequate through the rest of the slab depth. The deterioration can also be worse near joints and cracks, where moisture is allowed to infiltrate and saturate the concrete. To assess the extent of deterioration, cores should be taken and analyzed at representative distressed and nondistressed areas.

For pavements with low-severity freeze-thaw deterioration of the cement paste, treatment methods are recommended. Treatment methods can stop or at least slow the progression to moderate-severity levels, thus extending the life of the pavement. The recommended method for low-severity deterioration, especially for deterioration concentrated at the pavement surface, is the application of a surface sealer. The sealer should be applied to the entire pavement surface to slow the rate of deterioration and prevent further progression. Water-based and solvent-based silane sealers have been used effectively for such applications. Another feasible method is sealing joints and cracks. However, this method cannot completely eliminate the intrusion of water into the pavement system and is therefore limited in its effectiveness. Sealing joints and cracks can be more effective when used in combination with surface sealing.

For moderate-severity freeze-thaw deterioration, the best treatment method depends on the extent of deterioration. If the deterioration is limited to the pavement surface, such as exhibited by scaling or map cracking, only surface repair methods are necessary. Where the deterioration is limited to small areas along joints or other isolated locations, partial-depth or full-depth repairs are recommended. Diamond grinding can be used after any repairs to remove surface irregularities and to improve the serviceability of the pavement. The application of a surface sealer after diamond grinding should be considered to prevent the recurrence of deterioration.

Partial-depth repairs and diamond grinding are not recommended on pavements where freeze-thaw deterioration is not confined to the pavement surface. In such cases, rehabilitation methods that address the deterioration through the entire depth of the slab are required. For deterioration that is limited to transverse joints or other small isolated areas, full-depth repairs are recommended. For larger isolated areas, slab replacement is more cost effective. These methods address the full extent of deterioration. Where these methods are believed to be ineffective or where the number of repairs will be too costly, an alternative is to overlay the pavement.

Treatment methods should not be used on pavements with high-severity deterioration. For such deteriorated areas, rehabilitation methods that involve removal and repair of the deteriorated area are required. Deterioration confined to transverse joints and cracks can be repaired effectively with full-depth repairs. Full-depth repairs can also be used to repair small isolated areas; slab replacement is more cost effective for repairing larger isolated areas. When repairs become too abundant and costly, reconstruction is the only remaining alternative. Recycling of the pavement for aggregate offers potential savings and conservation of resources.

Freeze-Thaw Deterioration of Aggregate

Freeze-thaw deterioration of aggregate, commonly known as D-cracking, is caused by freezing and thawing of water absorbed into susceptible coarse aggregate particles. The treatment/rehabilitation method must focus on eliminating one or more of the conditions that cause freeze-thaw deterioration, specifically susceptible aggregates, freezing temperatures, or available moisture. Treatment of in-place susceptible aggregates is not a feasible alternative. And although placement of an overlay can reduce the number of freeze-thaw cycles by reducing the exposure to severe conditions, it is impossible to eliminate freeze-thaw cycling in harsh climates. Thus, treatment methods focus on eliminating the amount of available moisture in the pavement. Rehabilitation methods are generally considered a temporary fix to extend the life of the pavement until major rehabilitation is performed.

Available Treatment Methods for Freeze-Thaw Deterioration of Aggregate

Most treatment methods are designed to reduce the amount of moisture in the pavement system. Such methods include sealing joints and cracks, sealing the pavement surface, and retrofitting drains. Many question the effectiveness of any method designed to eliminate moisture. Although the amount of water from surface infiltration can be reduced, it can never be completely eliminated. In addition, the underlying subgrade commonly remains continually moist, even when the most elaborate drainage system is used. Furthermore, water can enter the pavement from other sources, such as laterally from ditches and upward from the groundwater table. Nonetheless, there is still 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.

Freeze-thaw deterioration of aggregate typically initiates at joints and cracks where water is allowed to infiltrate, so any measures that prevent the intrusion of water can be effective. Sealing joints and cracks will limit the amount of infiltrated water, although it will not completely eliminate infiltration. Moisture can still penetrate through well-sealed joints as well as from other sources.

Similar concerns are also valid for surface sealers. Their usefulness is questioned because freeze-thaw deterioration of aggregate typically initiates at the bottom of the slab and propagates upward. Nonetheless, surface sealers have been used to combat the effects of freeze-thaw deterioration. Laboratory 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). Similar results were obtained in the field. Another field experiment using silane sealers indicated mixed results, although they were found to be more effective on pavements with less deterioration (Engstrom 1994).

Another treatment method designed to reduce the amount of available moisture is the addition of retrofitted drainage, although its effectiveness is also questionable. An important consideration is the permeability of the layers beneath the slab. Studies have shown that if water cannot drain by gravity within the underlying layers to the edge 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). When used solely to address freeze-thaw deterioration, retrofitted drainage provides limited benefits and is not cost effective, mainly due to the inability to move infiltrated water to the drains. However, if other moisture-related distresses are present, such as pumping or corner breaks, retrofitted drainage is more cost effective as an all-inclusive treatment method.

Another treatment method is filling cracks with a bonding material such as an HMWM. This type of treatment strengthens the concrete by effectively "gluing" 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). On the same note, HMWM should only be applied at joints and cracks; there are no benefits to applying it to the entire pavement surface unless cracking is present throughout the pavement. Field experiments found that HMWM were effective for up to 18 months, which implies that reapplication at such intervals may be required.

Available Rehabilitation Methods for Freeze-Thaw Deterioration of Aggregate

Rehabilitation methods for addressing freeze-thaw deterioration of aggregate involve the removal and replacement of distressed concrete areas. Feasible repair methods include partial-depth repairs, full-depth repairs, overlays, reconstruction, and recycling. Some pavement engineers believe that repair methods are not effective means to repair freeze-thaw deteriorated areas because they do not address the cause of the distress. However, depending on the objective (such as to extend the life of the pavement), the repair methods can be effective, especially in the short term.

Freeze-thaw deterioration is often confined to transverse joints, and is typically more severe at the slab surface and bottom. In isolated cases, it has been treated using partial-depth patching in which the deteriorated surface is removed using cold milling. This procedure does nothing to restore the deteriorated bottom of the slab, but does restore rideability, at least in the short term. More commonly, full-depth repairs are used as an effective method of rehabilitation. However, they also create an additional joint, where deterioration is likely to appear within 5 years. Treating the joint and possibly the adjacent concrete with a surface sealer can help delay the onset of the deterioration. Even with preventive measures, full-depth repairs should be viewed only as a means to extend the life of the pavement.

The placement of an asphalt concrete (AC) overlay has been the most common form of rehabilitation on concrete pavements exhibiting freeze-thaw deterioration of aggregate (Schwartz 1987). One philosophy is that by covering the concrete, it will not be directly exposed to air and deicers, thus effectively reducing the number of freeze-thaw cycles. But an AC overlay 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. Even though an overlay decreases the number of freeze-thaw cycles, its placement can actually accelerate the rate of deterioration by increasing saturation (Janssen 1985; Janssen et al. 1986). Consequently, AC overlays may not be very effective when placed over a pavement affected by freeze-thaw deterioration and should be used cautiously unless the underlying concrete is rubblized.

An unbonded portland cement concrete (PCC) overlay is another rehabilitation option. Again, the overlay will not prevent freeze-thaw cycling in the underlying pavement. However, an unbonded PCC overlay can be more effective because its performance depends less on the condition of the underlying pavement; providing uniform support is the most important consideration. Rubblizing the existing pavement, ensuring that full rubblization has occurred, before placing the overlay will prevent continued deterioration from freeze-thaw deterioration, but will likely result in the need for a thicker overlay.

The final alternative is removal and reconstruction of the pavement. Of course, this alternative is most cost effective on badly deteriorated concrete pavements, where treatment methods are ineffective and repairs are too numerous and costly. Recycling of the deteriorated concrete pavement as aggregate for the new pavement offers a variation within the reconstruction alternative. With special design considerations to prevent the recurrence of freeze-thaw deterioration (e.g., using top aggregate size smaller than 19.5 mm [0.75 inches] and limiting the amount of recycled fines), recycling has proven to be an effective rehabilitation method.

Selection Guidelines for Freeze-Thaw Deterioration of Aggregate

As with any distress, the most appropriate treatment or rehabilitation method for pavements exhibiting freeze-thaw deterioration of aggregate depends on the extent and severity of deterioration. Table 20 provides some basic guidelines for selecting feasible repair alternatives based on the extent and severity of deterioration observed on the pavement section. These should be viewed as rough guidelines for selecting potential feasible alternatives. The severity of the distress can be assessed using the following guidance:

Table 20. Selection of feasible alternatives to address freeze-thaw deterioration of aggregate (Van Dam et al. 2002a).

Severity	**Extent**	**Feasible Alternatives**	**Comments**
Low	Corners	Seal the pavement Seal joints and cracks	Treatments should be aimed at limiting the amount of available moisture and delaying the progression to higher severity levels.
	Joints and cracks	Seal the pavement Seal joints and cracks	
Moderate	Corners	Seal joints and cracks Partial- or full-depth repairs Overlay	Limit moisture to prevent further deterioration. Full-depth repairs should be considered a temporary fix (extend life about 5 years). HMWM requires reapplication to be most effective (about every 18 months).
	Transverse joints and cracks	Seal joints and cracks Apply HMWM Partial- or full-depth repairs Overlay	
	Longitudinal joints	Seal joints Apply HMWM Overlay Recycling Reconstruction	Damage is too widespread to repair each area. Recycling of deteriorated pavements has proven to be a feasible alternative.
High	Corners	Full-depth repairs	Deterioration is too severe for treatment; deteriorated areas must be removed and replaced.
	Transverse joints and cracks	Full-depth repairs Recycling Reconstruction	
	Longitudinal joints	Recycling Reconstruction	Deterioration is too severe and too extensive for treatment or restoration.

- Low severity is characterized by tight cracks, no loose pieces or spalling, and no patching in the affected area. Staining on the surface may also be evident in the affected area.
- Medium severity is characterized by well-defined localized cracking and some small pieces may be loose or missing. Staining is likely to be observed in the affected area.
- High severity is characterized by a well-developed crack pattern and a significant amount of missing pieces. Patching may be evident in affected area. Staining will likely be evident in affected area.

In general, pavements exhibiting low-severity freeze-thaw deterioration of aggregate can be addressed through treatment methods because the deterioration is not creating a serviceability problem. Such methods may prevent further progression of freeze-thaw deterioration to higher severity levels and extend the pavement life. High-severity freeze-thaw deterioration, on the other hand, generally requires removal and repair of the distressed area. At this point, the pavement is too deteriorated, and treatment methods no longer offer an effective means of addressing the problem. For pavements exhibiting moderate-severity deterioration, the preferred method—treatment or rehabilitation—will further depend on the extent of deterioration.

The extent of deterioration also influences the specific treatment or rehabilitation method to be used. Freeze-thaw deterioration of aggregate typically initiates at the slab corners and progresses along the transverse joint. If deterioration is limited to these areas, the most effective methods are those that reduce the amount of available moisture at joints (for low-severity) or localized repairs (for higher severity). As freeze-thaw deterioration begins to progress along longitudinal joints and further into the slab interior, localized methods become less and less effective. Methods need to be employed that address the entire pavement, such as a surface sealer (low severity) or reconstruction (high severity). In the end, it is more cost effective to reconstruct the entire section than it is to conduct extensive full-depth patching (Hoerner et al. 2001).

Regardless of the extent of freeze-thaw deterioration, cores should be taken at representative locations throughout the pavement 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. 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.

For low-severity freeze-thaw deterioration of aggregate, treatment methods designed to limit the exposure to moisture will be most effective. An important point to remember is that freeze-thaw deterioration of aggregate commonly initiates at the bottom of the slab, where moisture is more readily available, and propagates upward, so methods designed to prevent surface infiltration will only be partially effective. In other words, they may reduce the rate of freeze-thaw deterioration but will never fully arrest its development. With that said, the best treatment method to address low-severity freeze-thaw deterioration is a combination of sealing/resealing joints and cracks and the application of a surface seal. Sealing joints and cracks will prevent excess moisture from infiltrating into the pavement and saturating the underlying layers. Meanwhile, the surface sealer will prevent the absorption of water through the surface; although it may require reapplication due to traffic wear. The surface sealer should be placed over the entire pavement surface to help slow the rate of deterioration and to prevent further deterioration. A water-based or solvent-based silane sealer is recommended.

For moderate-severity freeze-thaw deterioration of aggregate, the application of an HMWM has shown favorable results. To be effective, the cracks should be wide enough for the material to penetrate, which is the reason HMWM is not recommended for low-severity deterioration. Where the deterioration is limited to transverse and longitudinal joints and cracks, the application of an HMWM is recommended. The material should be applied only to cracked areas and will need to be reapplied at approximately 18-month intervals. The only other potential treatment method is sealing joints and cracks. The use of surface sealers is not recommended for deterioration that has progressed to moderate-severity levels. In terms of rehabilitation methods, full-depth repairs have been found to be the most cost-effective method for addressing moderate-severity freeze-thaw deterioration that is confined to transverse joints and cracks (Schwartz 1987). Full-depth repairs are also recommended in conjunction with treatment methods.

Where freeze-thaw deterioration has progressed to longitudinal joints or throughout the entire slab, full-depth repairs will no longer be cost effective. In such cases, methods that address the entire pavement need to be employed (e.g., overlay, reconstruction, or recycling). An AC overlay will provide a smooth-riding surface and years of service but will not prevent the further progression of freeze-thaw deterioration. This continued deterioration must be addressed in design by either placing a thicker overlay than normal or accepting a shorter service life. Other overlay options, such as rubblizing or recycling, are generally not cost effective for moderate-severity deterioration (Hoerner et al. 2001). A more effective solution is to allow the pavement to deteriorate (i.e., wait until distress progresses to high severity) before conducting such extensive rehabilitation.

For high-severity freeze-thaw deterioration of aggregate, treatment methods are not effective. The only applicable methods are those involving the removal and replacement of badly deteriorated areas, of which full-depth repairs are the most effective method. However, this method should be viewed as a temporary fix to extend the life of the pavement and must consider the overall pavement performance. For example, it is impractical to conduct extensive full-depth patching when the entire pavement will require reconstruction within a few years.

On pavements with freeze-thaw deterioration at nearly every joint (some of which is high severity), the most effective alternative is to either reconstruct or rubblize and overlay the entire pavement. At this point, treatment and restoration methods will no longer be effective. As a form of reconstruction, recycling of the damaged pavement as aggregate for the new pavement can be effective if appropriate measures are taken to prevent the recurrence of freeze-thaw deterioration. Another alternative is to rubblize the deteriorated pavement for use as a base course for a new pavement. This alternative involves other considerations, such as grade changes and overhead clearances, which must be addressed as part of the overall selection process.

<u>Alkali–Silica Reactivity</u>

ASR is a reaction between alkalis in the cement paste and reactive silica found in some aggregate sources. The reaction forms a gel product that expands in the presence of moisture. The expansion initially appears on the pavement surface as irregular, map-like cracking and can ultimately lead to joint spalling, blowups, and other pressure-related distresses. ASR is rarely confined to isolated areas such as joints and cracks but rather occurs throughout the entire pavement. Consequently, treatment and rehabilitation methods must address the entire pavement area.

Available Treatment Methods for Alkali–Silica Reactivity

The factors controlling ASR are the amount and properties of reactive silica, the amount of available alkali, and the amount of available water (Mindess and Young 1981). The amount and properties of reactive silica and available alkalis (assuming no external sources) are controlled by the constituent materials (namely the aggregate and cement) and cannot be altered. As discussed for other MRD, methods designed to limit the amount of available water will have limited effectiveness. The use of such methods 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.

A promising alternative for addressing ASR is treatment with lithium compunds. Lithium compounds were first found to be an effective treatment in 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 but have yet to receive widespread use in the field. There are currently several on-going experimental projects being conducted to evaluate their effectiveness, and early results have been favorable (Stark et al. 1993; Johnston 1997). The major limitation for field applications is achieving penetration of the lithium solution through the depth of the 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. Of the treatment solutions, LiOH solutions were more effective in controlling expansion than Li_2CO_3 and LiF solutions. However, 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 (Johnston 1997). That study avoided the use of LiOH due to safety concerns (it is highly alkaline) and ability of OH ions to accelerate the reaction.

Another treatment method is the application of HMWM, which is designed to fill and bond cracks in order to strengthen the pavement. Although HMWM can only penetrate surface cracks, the map-like cracking pattern produced by ASR typically extends only 50 to 75 mm below the surface. Studies have shown that HMWM penetrates cracks up to 50 mm deep (maximum depth of the surface cracks) and can initially reduce midslab and decrease joint deflections (Stark et al. 1993).

Available Rehabilitation Methods for Alkali–Silica Reactivity

ASR does not require complete saturation of the concrete to produce swelling and expansion; even high relative humidity levels (greater than 80 percent) have been shown to produce cracking from expansive ASR. Map cracking exhibited in localized areas of the pavement will eventually progress throughout the entire pavement area. Therefore, restoration techniques that only address a specific area are not effective for pavements exhibiting ASR. Localized repair methods such as full-depth repairs will only provide temporary solutions to the problem. Therefore, restoration techniques are only recommended to repair isolated areas of severe deterioration to maintain serviceability and smoothness (i.e., buying time until more extensive rehabilitation efforts). An AC overlay is more applicable for extensive ASR. An overlay can help slow the deterioration rate by decreasing the moisture gradient (between the top and bottom of the slab), which promotes more uniform expansion due to ASR through the slab depth.

The only applicable rehabilitation methods for fully addressing ASR distresses are rubblizing, recycling, and/or reconstruction. These methods involve the complete destruction of the slab and, in the case of the latter two alternatives, removal of the pavement. Other methods do not address the problem, and ASR damage will continue to progress in areas that are not repaired. With special considerations during mix design, such as the use of pozzolans, recycling of ASR-damaged concrete has been used successfully as aggregate for new concrete.

Selection Guidelines for Alkali–Silica Reactivity

Table 21 presents general guidelines for selecting feasible alternatives to address ASR. Unlike other MRD, these guidelines are based solely on the severity of the distress. The extent of deterioration is not included because ASR affects the entire pavement area and not just isolated areas such as joints (although the damage can be worse in isolated areas). ASR initially appears as map cracking and the presence of an exudate is possible. Over time, exudate is almost always observed and expansion-related distress becomes evident. Treatment methods are best suited for pavements exhibiting low- to moderate-severity distress. The severity of the distress can be assessed using the following guidance:

- Low severity is characterized by fine hairline map cracks, no scaling, no evidence of expansion, and no patching in the affected area. Staining may be observed in the vicinity of cracks.
- Medium severity is characterized by a readily visible network of well-defined, tight map cracks. Some minor evidence of expansion may be observed. Exudate may be visible in the cracks.
- High severity is characterized by a well-developed pattern of open cracks. Spalling and scaling, as well as patching may be evident in some areas. Exudate is typically evident in cracks and evidence of expansion-related distress should be observed.

Rehabilitation methods that address isolated areas are generally ineffective and are only recommended on pavements in which the deterioration is creating a safety problem. Otherwise, treatment methods should be used. If treatment methods are ineffective, then the best option is to let the pavement live out its life and then reconstruct the pavement.

Table 21. Selection of feasible alternatives to address ASR (Van Dam et al. 2002a).

Severity*	Feasible Alternatives	Comments
Low	Apply lithium salts Apply HMWM	Objective is to prevent or delay further deterioration. Measures designed to prevent access to moisture will likely be ineffective, especially in wet climates.
Moderate	Apply lithium salts Apply HMWM Overlay	Delay or slow the progression of deterioration.
High	Rubblization Recycling Reconstruction	Deterioration is too severe for treatment or restoration.

* The extent of deterioration is not considered because ASR generally occurs throughout the entire slab.

A variety of feasible treatment methods are available for addressing low-severity ASR. Due to the relatively recent understanding of ASR, the long-term field performance of the treatment methods is uncertain. However, several methods have been found to be effective in laboratory testing and in short-term field experiments. Two of the more promising methods are the application of lithium salts and the application of HMWM. Either of these methods is recommended for low-severity ASR. With both methods, the key to success is to achieve penetration through cracks and into the pavement by ensuring adequate material is applied in a uniform manner.

The recommended treatment method for moderate-severity map cracking due to ASR is the application of HMWM. HMWM penetrates cracks in the pavement and strengthens the concrete. The benefits are not reduced due to the higher severity cracks, as the wider cracks will allow easier access and penetration into the cracks. However, the effectiveness can be reduced due to traffic wear and environmental exposure, so reapplication at about 18-month intervals is often necessary. The use of lithium compounds may also be effective. And although an AC overlay does not directly address the problem, it can be an effective method of improving serviceability.

For pavements that exhibit high-severity damage caused by ASR, the only viable alternative is to reconstruct the pavement. At this point, the deterioration is too severe for treatment methods, and because ASR affects the entire pavement area, the deterioration is too extensive for localized repairs. With the inclusion of pozzolans, crushed concrete from pavements with ASR can be reused successfully.

<u>Sulfate Attack</u>

Deterioration due to sulfate attack is generally attributed to chemical decomposition of certain cement hydration products and/or the formation of an expansive reaction product, ettringite (DePuy 1994). The development of additional ettringite, which is considered the main destructive force in sulfate attack, can result in significant volume expansion and cracking. Deterioration due to sulfate attack first appears as cracking near joints and slab edges that can also progress to fine longitudinal cracking throughout the slab. In same cases, sulfate attack has been characterized by a series of closely spaced, tight map cracks with wider cracks appearing at regular intervals. The variation in manifestations makes it easy to confuse sulfate attack with other MRD types.

Sulfate attack is commonly subdivided according to the source of the sulfate ions. External sulfate attack results from the penetration of sulfate ions from outside sources (e.g., groundwater, seawater, soil, or impurities in chemical deicers) into the concrete. Internal sulfate attack occurs when the source of the sulfate ions is internal, either from one of the constituents or due to the decomposition of primary ettringite as a result of high curing temperatures. Although the source of the sulfate ions differs, the mechanisms and the treatment methods are the same for internal and external sulfate attack.

Available Treatment Methods for Sulfate Attack

There are still many unknowns as to the cause and impact of sulfate attack. Likewise, methods for effective treatment have not developed to the level of methods used to address other MRD. Treatment methods for sulfate attack must either prevent sulfate ions from penetrating into the concrete or disrupt the reaction to limit expansion and/or decomposition. The first method, preventing the penetration of sulfate ions, is only viable if the sulfate ions are from an external source that can be stopped. Nothing can be done if the sulfate ions are contained within the concrete.

Like most MRD types, sulfate attack requires the presence of moisture to transport sulfate ions to the reaction sites, so methods designed to remove excess water from the pavement also offer feasible alternatives. However, the reaction does not require complete saturation of the concrete; a relative humidity of 80 to 90 percent is all that is required to fuel the reaction (Thaulow et al. 1996). As a result, methods to limit the amount of available water in the pavement system, such as sealing joints and cracks, are not very effective.

Sealing the pavement can help prevent the penetration of sulfate ions from external sources. Practically, however, only the pavement surface can be sealed, and the sources of sulfate ions (groundwater, seawater, and soils) more often than not penetrate from the bottom of the slab. For this reason, the benefits of sealing are questionable.

The addition of chloride ions is a possible treatment method. Ettringite has been found to dissolve in the presence of chloride ions, particularly NaCl (Attiogbe et al. 1990; Marks and Dubberke 1996). 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. Further investigations into this initial expansion are currently being conducted. The potential for ASR should be investigated before using NaCl to treat sulfate attack, as the NaCl can further increase ASR potential and contribute to corrosion of reinforcement. The excess alkali can increase hydroxyl ion concentrations and possibly convert an otherwise innocuous cement-aggregate combination into a deleterious one (Stark 1994).

Available Rehabilitation Methods for Sulfate Attack

Rehabilitation methods for addressing damage due to sulfate attack involve the removal and replacement of the material. These methods are most applicable for repair of areas of moderate- to high-severity distress. Where the deterioration is confined to corners or along transverse joints and cracks, full-depth repairs offer a feasible alternative. Partial-depth repairs are not recommended because the deterioration is often worse at the bottom of the slab. Although effective, full-depth repairs should be viewed as temporary fixes (about 5 years). Placement of full-depth patches creates two new joints where there was only one joint previously, thus creating another avenue for water to infiltrate into the pavement.

When the damage becomes more extensive, rehabilitation methods that address the entire pavement area must be employed. One option is the placement of an overlay to cover the extent of the deterioration. Either an AC overlay or an unbonded PCC overlay is acceptable; bonded PCC overlays are not recommended over pavements exhibiting distress caused by sulfate attack. In either case, the design of the overlay should consider the continued deterioration of the underlying pavement because the overlay will not stop the mechanism of sulfate attack. Designers should take extra measures to ensure the expected life is achieved or must accept the fact that the overlay will not provide the normal expected life.

When the damage becomes too severe, overlays will no longer be effective because the cost of preoverlay repair becomes too great. If the deteriorated areas are not repaired, the overlay will fail prematurely. At this point, the alternatives are limited to rubblize and overlay, recycling, or reconstruction. When constructing the new pavement, consideration should be given to limit the intrusion of sulfate ions from groundwater or soil sources.

Selection Guidelines for Sulfate Attack

Table 22 provides a summary of the feasible alternatives to address sulfate attack. Sulfate attack has many manifestations, but most commonly first appears on the pavement surface as fine cracking near joints and slab edges or as map cracking over the entire surface. As sulfate attack progresses, spalling will ensue and in some cases, a complete disintegration of the mortar fraction occurs. Distress manifestations for sulfate attack are similar to other distresses and thus an important step in the selection process is ensuring that the pavement is indeed experiencing sulfate attack based on the proper application of laboratory procedures presented in guideline II. The severity of the distress can be assessed using the following guidance:

Table 22. Selection of feasible alternatives to address sulfate attack (Van Dam et al. 2002a).

Severity	Extent	Feasible Alternatives	Comments
Low	Corners	Seal pavement (external) Seal joints and cracks	Reaction does not require complete saturation, so means to reduce the amount of moisture are not as effective. Sealing the pavement is an alternative if source of sulfate ions is external.
	Transverse and longitudinal joints	Seal pavement (external) Seal joints and cracks	
	Entire slab	Seal pavement (external)	
Moderate	Corners	Full-depth repairs	Full-depth repairs should be considered a temporary fix; deterioration may continue adjacent to the patch.
	Transverse joints and cracks	Full-depth repairs	
	Longitudinal joints	Seal joints	Damage is too widespread to repair each area.
	Entire slab	Overlay	
High	Corners	Full-depth repairs	Deterioration is too severe for treatment; deteriorated areas must be removed and replaced.
	Transverse joints and cracks	Full-depth repairs	
	Longitudinal joints	Rubblize and overlay Recycling Reconstruction	Deterioration is too severe and too extensive for treatment or restoration.
	Entire slab	Rubblize and overlay Recycling Reconstruction	

- Low severity is characterized by fine hairline map cracks, no scaling, no evidence of expansion, and no patching in the affected area. The pattern may be isolated to joints or slab edges or present over the entire slab surface. Staining may be observed in the distressed area.
- Medium severity is characterized by a readily visible network of well-defined, tight map cracks either isolated to joints or slab edges or present over the entire slab surface. Some minor spalling, scaling, and/or disintegration of the mortar fraction may be observed. Evidence of exudate accompanied with staining may be visible.
- High severity is characterized by a well-developed pattern of open cracks, accompanied by widespread spalling, scaling, and/or disintegration of the mortar fraction. Patching may be evident in affected areas. Evidence of exudate accompanied with staining may be visible. Evidence of expansion may be observed.

Methods for treating low-severity deterioration are sealing the pavement and sealing joints and cracks. The effectiveness of these methods is limited because none will completely eliminate the progression of sulfate ions into the pavement surface, nor address the influx from beneath the slab.

For moderate-severity deterioration, where cracking and spalling are limited to transverse joints and cracks, full-depth repairs are a cost-effective means to restore serviceability and extend the life of the pavement. If the deterioration is more extensive, an overlay will be more cost effective, as the number of full-depth repairs required will become too costly.

If the deterioration has progressed to high severity, only methods that involve repair of the deteriorated areas will be acceptable. Full-depth repairs are recommended for deterioration confined to transverse joints and cracks. Otherwise, recycling or reconstruction will be the most cost-effective alternative.

Summary of Treatment and Rehabilitation Methods

This section of the chapter presented information on selecting feasible treatment or rehabilitation option to address MRD in concrete pavements. Treatment methods focus on eliminating or reducing the 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 guidance for each distress type is 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.

MATERIALS AND MIX DESIGN FOR PREVENTION OF MRD IN CONCRETE PAVEMENTS

This section of the chapter considers specific factors that have a direct influence on preventing the MRD types observed in the pavements under study. It is not designed to supplant existing mix design and construction practices, such as those advocated by the Portland Cement Association (Kosmatka et al. 2002), the American Concrete Institute (ACI) (1991), or those currently used by MDOT, but instead supplement them through increased consideration of long-term concrete durability.

To construct durable concrete pavements, the selection of constituent materials, mixture design, and construction practices should be approached from a holistic point of view. Mehta (1997) presents this concept in a recent paper, stating "that current theories on the mechanisms responsible for deterioration of concrete due to various causes are based on a reductionistic approach to science." This approach tries to understand a complex system by reducing it to parts, considering only one aspect of the problem at a time. As a result, a given test method is focused only on a single attribute, failing to consider the system as a whole. The need for a holistic approach in addressing concrete durability is evident when one considers how often two or three MRD mechanisms appear to be at work simultaneously in a distressed concrete specimen. This makes it nearly impossible to separate the actual "cause" of distress from opportunistic distress that became manifest only after degradation had already begun. By adopting the holistic approach in which concrete integrity and watertightness is the goal, more durable concrete will be produced.

In general, to achieve durable concrete pavements, inherently durable materials must be selected, combined, and constructed to create concrete that is relatively impermeable and physically and chemically stable. This is normally accomplished by carefully selecting the constituents (aggregate, cementitious materials, admixtures, water) based on experience and testing. For example, MDOT realized through experience that certain carbonate aggregates were susceptible to aggregate freeze-thaw deterioration and thus implemented a rigorous testing program using standardized cyclic freeze-thaw testing (MTM 115). Largely as a result of this effort, aggregate freeze-thaw deterioration does not seem to be a problem in recently constructed concrete pavements. Once materials are selected, a mixture must be designed that not only possesses adequate strength, but also durability and economy. This is accomplished by keeping the *w/cm* below 0.50 (0.42 to 0.45 is pretty common) while minimizing the paste fraction through aggregate grading and increasing aggregate size. The MDOT Portland Cement Concrete Grade P1 (Modified) (MDOT 2001) reflects this philosophy by blending two coarse aggregate gradations with the maximum aggregate size set at 62.5 mm (2.5 inches). Once the materials and mixture design have been selected, it has been found advantages to conduct further durability tests on the job mixture itself. The final step is to ensure that during all phase of construction, practices are followed that will result in a high quality product.

There are a number of accepted works on concrete mixture design, construction, and durability such as *The Design and Control of Concrete Mixtures* (Kosmatka et al. 2002), *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete* (ACI 1991), the *Guide to Durable Concrete* (ACI 1992), and *Durability of Concrete* [Transportation Research Board (TRB) 1999]. In addition, the FHWA guidelines (Van Dam et al. 2002a) provide in-depth discussion on how to design and construct durable concrete pavements. In this report, specific recommendations regarding the prevention of the MRD identified in this study will be provided. This discussion is based largely on the information provided in the FHWA guidelines (Van Dam et al. 2002a).

Controlling Specific Types of MRD

Paste Freeze-Thaw Deterioration

Although paste freeze-thaw deterioration was not directly implicated in any of the observed deterioration, it was noted that the air-void systems were marginal in four of the test sites evaluated). It is common enough that it is worthwhile for MDOT to review methods needed to ensure that an adequate air-void system is obtained. The only practical technique available to avoid paste freeze-thaw damage in saturated concrete located in a freeze-thaw environment is to entrain air voids of the proper size and spacing in the concrete matrix. 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. But interactions with other mixture constituents can negatively affect the performance of the admixture with undesirable results.

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 air during slipform paving is not uncommon. Also, the measurement of overall air content does not separate entrapped air from entrained air.

For these reasons, it might be advantageous to MDOT to consider assessment of the air-void system in hardened concrete during mix design and as part of the construction process. This is normally done microscopically using procedures described in ASTM C 457. This practice is currently the only accepted means to determine if the air-void system characteristics are adequate to protect the paste from freeze-thaw deterioration. MDOT currently has the capacity to conduct this test method in-house, but would need to be careful not to overwhelm the technical staff since the test is time consuming. The use of an automated air-void analysis system can significantly reduce the time and labor required to conduct this testing, but it must measure all the relevant air-void system parameters, not just total air. In addition to currently available automated systems, new techniques are becoming available that might simplify this process (Peterson et al. 2001a, Peterson 2001b, Peterson et al. 2001c).

Aggregate Freeze-Thaw Deterioration

Aggregate freeze-thaw deterioration was observed to be the primary mechanism in three of the oldest sections evaluated (Test Sites Nos. 12, 22, and 28) and is also likely contributing to the demise of the Clare Test Road (Test Sites Nos. 26A, 26B, and 27). 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. Although this approach on the outside may seem attractive, it is not always a practical solution in Michigan where otherwise high quality susceptible carbonate aggregates represent a significant portion of the continually diminishing aggregate resources.

Therefore, it has been necessary for these marginal aggregate sources to undergo benefaction to reduce the susceptibility to this MRD. Benefaction generally takes four many forms as illustrated in Figure 20, with three methods having been used with variable success. The first method attempts to separate susceptible aggregate from nonsusceptible aggregate using the aggregate bulk specific gravity. This method is frequently used in Michigan to separate out lighter aggregate particles that are generally deleterious and have been implicated in popouts and spalling. It has been found in some cases that aggregates having lower bulk specific gravities were less resistant to freeze-thaw deterioration and that this difference could be used to separate aggregate using heavy-media separation (Schwartz 1987). With the elimination of the lighter aggregate particles, the remaining aggregates are more resistant to freeze-thaw deterioration.

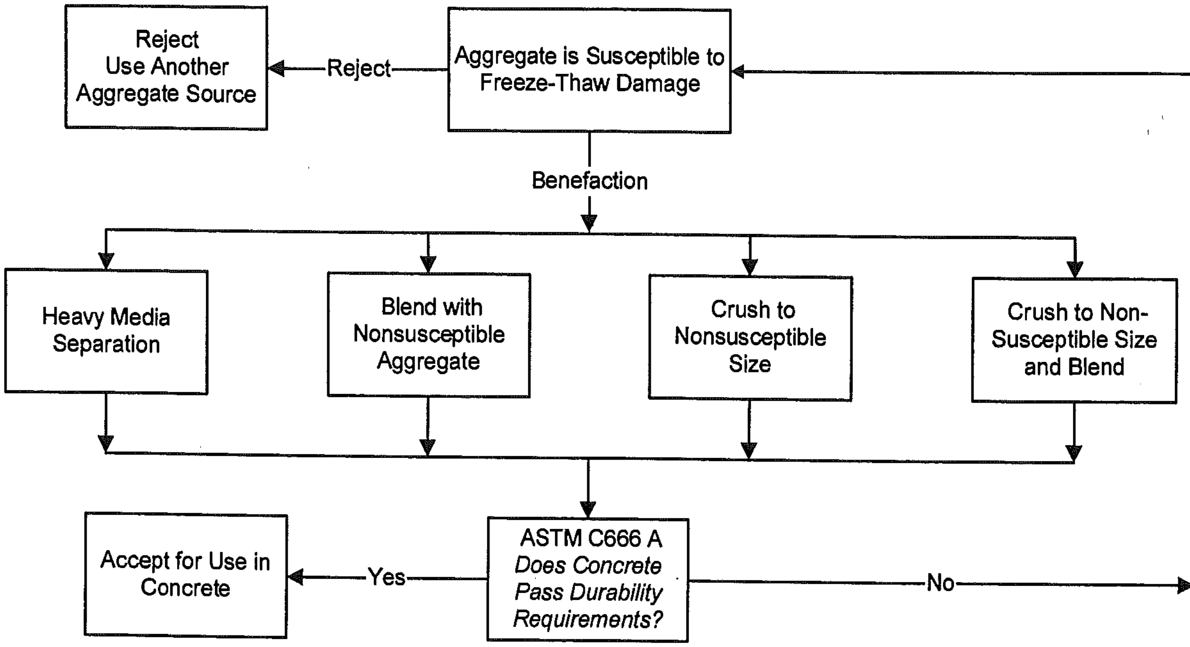


Figure 20. Benefaction techniques for mitigation of aggregate freeze-thaw deterioration (Van Dam et al. 2002a).

The second method to improve the freeze-thaw resistance of an aggregate source is to blend it with an aggregate source that is known to be resistant to freeze-thaw distress. Blending simply dilutes susceptible aggregate with non-susceptible aggregate, increasing the overall freeze-thaw resistance 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). Further, deleterious particles near the pavement surface will swell and/or fracture, causing popouts and spalls at joint edges.

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 including Michigan 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, it has been found to be effective to reduce the maximum aggregate size to 19.5 mm (0.75 inches), but in some cases the maximum aggregate size has been reduced to as little as 12.5 mm (0.5 inches) to effectively mitigate freeze-thaw damage. Because of the ease of applying such criteria, it has become common practice in Michigan to reduce the maximum aggregate size to 19.5 mm (0.75 inches) even if the aggregate is not freeze-thaw susceptible, since this also helps workability. 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, which is of particular importance in the longer jointed reinforced concrete pavements used in Michigan.

With the desire to use the largest maximum aggregate size possible, as is reflected in MDOT's Portland Cement Concrete Grade P1 (Modified) (MDOT 2001), 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, nonsusceptible sizes, and then blended with larger aggregate obtained from a nonsusceptible source. This will provide the best performance at minimal additional cost, as only larger size fractions need to be purchased and shipped. It is noted that shipping great distances can add significant additional cost to the project.

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 MTM 113, 114, and 115. Although the test requires expensive equipment and considerable time, these tests can be used to establish the aggregate freeze-thaw resistance in advance of construction, and the results correlated with a more rapid test such as ASTM C 295 to monitor consistency of the aggregate source. It is evident that MDOT's use of MTM 115 has significantly reduced the occurrence of aggregate freeze-thaw deterioration in the last two decades. Test results from MTM 115 could now be better correlated to field performance to help fine-tune the acceptance criteria for pavements of different levels of service, further increasing the usefulness of this test method.

Alkali–Silica Reaction

One of the major conclusions of this study is that ASR is a larger problem in some regions in Michigan than originally thought. In pavements containing slag coarse aggregate, the chert fractions in the fine aggregate were observed to be deleteriously reactive. It is therefore recommended that efforts be focused on addressing ASR in the future to better understand the full extent of the problem.

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 "zerotolerance" 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 multilane, 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.

Regardless of the approach used, a variety of mitigation techniques are available. One obvious method is to reject the aggregate source in favor of aggregate not containing reactive silica. This is particularly true if the aggregate is found to be highly reactive, in which case mitigation methods may not be effective. One method to consider in such a case is to blend the aggregate containing highly 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 as the remaining reactive silica reacts more thoroughly with the available alkalis (ACI 1998). Other mitigation techniques can be employed in combination with blending if the potentially deleterious aggregate supply must be used for practical reasons. 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). In Michigan, heavy media separation might be a good option for some fine aggregate sources since the reactive chert has a specific gravity significantly less than the quartz.

A very common approach to mitigation of ASR is to reduce the available alkalis in the pore solution. The largest contributor of alkalis is portland cement, although pozzolans, ground blast furnace slag (GBFS), mixing water, some chemical admixtures, and some aggregates can potentially contribute alkalis as well (ACI 1998). Typically, the use of low alkali cement is considered effective in reducing the total alkalis in the mixture. ACI (1992) notes that low alkali cement (maximum of 0.60 percent equivalent Na_2O) should be specified if ASR potential exists. This is emphasized by Stark (1994) who states that the higher the alkali content of the cement, the greater the expansion due to reactivity. But Stark (1994) also mentions that simply specifying a cement with an equivalent alkali content less than 0.60 percent is not in itself sufficient to guarantee that expansive ASR will not occur when highly reactive aggregate is present in sufficient quantity. Further, if the cement content is high, even low alkali cement will contribute significant total alkalis to the mixture.

As mentioned, the cement is only one potential source of alkali in the concrete. Other potential sources 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 mixture (Gress 1997). Gress cites studies from Germany and Canada that report great success in controlling ASR by limiting the total alkali content in the mix to 3.0 kg/m^3 Na_2O equivalent. In the proposed Canadian standard (Fournier et al. 1999), the total alkalis are limited to a maximum of 3.0 kg/m^3 Na_2O equivalent for mild prevention to 2.2 kg/m^3 and 1.7 kg/m^3 Na_2O equivalent for moderate to strong preventative action, respectively. These criteria was applied in this study using extracted alkali contents and although some relationships were found between extracted alkali contents and deterioration, the limits set in the Canadian work would not be adequate to protect the affected pavements in this study. This is an area that needs special investigation since there seems to be a close association between deleterious ASR in the chert constituent of the fine aggregate and the presence of slag coarse aggregate.

Gress (1997) states that in areas with high ambient temperatures and highly reactive aggregate, that in addition to limiting alkalis, the use of other mitigation techniques may be necessary. These include (Gress 1997):

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

Blended hydraulic cements must conform to ASTM C 595 or ASTM C 1157 to control ASR. It has been reported that blended hydraulic cements may be very effective in controlling expansion due to ASR due to better fineness and dispersion that occurs as a result of intergrinding (ACI 1998). If raw or calcined natural pozzolans are being used, ASTM C 618 must be met. Recommendation regarding the use of supplementary cementitious materials, including fly ash, GBFS, and silica fume can be found in both the proposed AASHTO Guide Specifications (AASHTO 2000) and the proposed Canadian standards (Fournier et al. 1999), as well as in the ACI *State-of-the-Art Report on Alkali-Aggregate Reactivity* (ACI 1998). In general, Class F fly ash has been found to be more effective in mitigating ASR than Class C fly ash, requiring 15 to 30 percent, by mass, of cementitious material compared to greater than 30 percent, by mass, of cementitious material, respectively (ACI 1998). Additionally, the alkali content of the fly ash must be controlled, and it has been found that Class C fly ash may release a larger portion of their total alkalis into the concrete (Lee 1989). These results have been confirmed in a limited way in this study, in that the test sites containing Class F fly ash (Test Site 4A), no deleterious ASR was observed even though it was actively destroying a similarly constructed pavement constructed with Class C fly ash (Test Site No. 4)

Another mitigation technique that is showing increasing promise is the use of lithium-based additives. 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. 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 cannot 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 GBFS 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 GBFS 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 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 testing 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, using supplementary cementitious materials, or adding lithium compounds. Other methods are also available, including aggregate benefaction.

Sulfate Attack

The potential sulfate attack observed in this study is quite unique. Without a more detailed study of the factors contributing to the observed distress, only a hypothesis can be forwarded at this time. What is known is that unique deterioration mechanisms seem to be at work in some concrete pavements constructed with slag coarse aggregate. The deterioration is manifest as a complete breakdown of the concrete matrix, generally in the vicinity of joints. Microscopically, there is strong evidence of calcium sulfide dissolution near the contact zone with the hydrated cement paste, a preponderance of calcium hydroxide in the hydrated cement paste, and secondary ettringite filling adjacent voids and cracks. Deleterious ASR associated with the chert constituent in the fine aggregate is also common in these deteriorated sections, making it very difficult to determine the exact nature of the deterioration and the relationship that may exist between ASR and the dissolution of the calcium sulfide.

The hypothesis that is emerging as a result of this study is that in deteriorated slag pavements, the calcium sulfide that is in contact with the pore solution in the hydrated cement paste is undergoing an oxidation reaction resulting in dissolution. The calcium is precipitating as calcium hydroxide in the vicinity of the slag particles and the sulfate reacts with aluminate resulting in a type of internal sulfate attack. The hypothesized process is complex with multiple variables impacting the rate at which it might occur, or even if it occurs at all. Obviously, this process in not well enough understood at this time to even clearly implicate the calcium sulfide as being part of the observed deterioration, but enough evidence exist (elevated sulfate contents, observations of evacuated calcium sulfide dendrites, and the presence of considerable amounts of ettringite filling cracks and voids) to support a more in-depth analysis of this hypothesized phenomenon.

As a result of limited knowledge, it might be premature to make significant changes to the concrete mixtures to address sulfate attack. At the same time, it is worth presenting potential changes so they might be considered for future evaluation. Thus, the following general recommendations are made regarding the prevention of sulfate attack.

Concrete that is resistant to sulfate attack is of high quality and relatively impermeable. 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 (1995) gives recommendations for general chemical attack resistance. He recommends 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.

Many researchers have found a link between cement properties and resistance to 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. In moderate to severe exposure levels, 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.

DePuy (1994) reports that using cement low in C_3A will generally decrease sulfate attack susceptibility, but exceptions exist where low C_3A cements show poor resistance to sulfate attack while some cements high in C_3A were observed to have good sulfate resistance. He recommends 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. In ASTM C 452, mortar bars are made with portland cement and gypsum in such proportions that the SO_3 content is 7 percent by mass. After mixing and casting, the mortar bars are cured under very controlled conditions. The initial length measurement is made at 24 hours, and the specimen is then water cured at 23 °C. A second measurement is made at 14 days, and the change in length is reported. The test can be extended for longer periods of time. The maximum allowable expansion for an ASTM C 150 Type V cement is 0.040 percent at 14 days.

In ASTM C 1012, mortar bars are prepared and immersed in a sulfate solution, and the resulting expansion measured. The cementitious material used can be portland cement, or blends of portland cement and fly ash or slags, or blended hydraulic cements. The mortar bars are immersed in the sulfate solution after attaining a compressive strength of 20 MPa. A standard exposure solution containing Na_2SO_4 can be used, or another sulfate solution simulating anticipated field conditions might be substituted. Length measurements are made at 1, 2, 3, 4, 8, 13, and 15 weeks, and at selected intervals thereafter depending on the observed rate of length change. The allowable expansion at 180 days is 0.10 percent for ASTM C 595M Type IS (MS), IP(MS), IS-A(MS), and IS-A(MS) cements.

For very severe exposure, a Type V with added pozzolan or slag having a demonstrated ability to improve sulfate resistance must be used. Slag and pozzolans have a beneficial effect by reducing the permeability of the paste and by minimizing the amount of CH present. Reducing the amount of CH in the cement paste contributes to 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 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. 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. 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 with the actual materials to be used in the concrete. These general recommendations regarding the use of Class F fly ash have been borne out in this study. The single biggest difference between a durable concrete section constructed with slag coarse aggregate (Test Site No. 4A) and a deteriorating section (Test Site 4) is that the durable section section used a Class F fly ash whereas the deteriorating section used a Class C fly ash. This alone is not clear evidence of the role of fly ash, but it makes the use of Class F fly ash attractive in near term future projects until further study can ferret out the true relationship between fly ash and deterioration.

The replacement of portland cement with GBFS 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.

The sulfate resistance of concrete is decreased through the addition of calcium chloride, which is a common accelerating admixture. It therefore should not be added to concrete subjected to severe or very severe sulfate exposure conditions (ACI 1992).

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 GBFS 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 1992). 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 slags 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.

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 alkali–aggregate reactivity, 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 for 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.

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.

In summary, it is currently unclear what is the best approach to prevent sulfate attack because the distress is not well understood in Michigan's pavement concrete. It is thought that the following factors, the use of low alumina cements, low calcium pozzolans, and low permeability concrete, will contribute to sulfate resistance. 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. Perhaps the simplest approach that can be immediately implemented is to require the use of Class F fly ash when slag coarse aggregate is being used until the problem is better understood. Further research into the potential for internal sulfate attack is therefore warranted.

Summary of Prevention of MRD

MDOT has made significant improvements in ensuring the durability of concrete pavements. One good example of improvement is the recognition that aggregate freeze-thaw deterioration was a serious problem, properly identifying the cause of the problem, and then implementing a process based on laboratory testing (MTM 115) that ensure that nonsusceptible aggregates are used in pavement construction. Although not without controversy (primarily because it is viewed by some as being too restrictive), this process has been an unequivocal success in mitigating aggregate freeze-thaw deterioration in newly constructed pavements.

MDOT's current evaluation of the Portland Cement Concrete Grade P1 (Modified)(MDOT 2001) is another good example of efforts being made to ensure that pavements designed for the highest level of service are of the highest quality. The Grade P1 (Modified) concrete mixture incorporates many of the recommendations made in the previous discussion, including the use of blended coarse aggregate grading with large maximum aggregate size to minimize paste demand, a *w/cm* ratio less than 0.45, a maximum allowable cement content of 335 kg/m^3 (565 lbs/yd^3), and not permitting the use of Class C fly ash. It seems clear that such an approach would lead to overall improvements in durability regardless of the materials being used.

CHAPTER 7.0 — CONCLUSIONS AND RECOMMENDATIONS

This final chapter of the report summarizes the conclusions of the study and then makes recommendations regarding future research needs.

CONCLUSIONS

Based on the work conducted in the course of this study, the following conclusions can be drawn:

- Based on visual inspections of the pavement network, MRD continues to be manifest in in-service pavements throughout the State of Michigan. The manifestations include pattern cracking, scaling, spalling, delamination, joint deterioration, and staining. In the most severe cases, the ability of the pavement to service traffic has been compromised and rehabilitation is needed to restore ride quality. In pavements that have yet to carry the design traffic, this results in unexpected costs associated with premature failure of the structure at great cost to MDOT.
- Structural analysis of pavements affected by MRD, including compressive strength and nondestructive deflection testing, indicates that a slight loss of structural capacity may be incurred as a result of the deterioration. The results are not conclusive, but on average compressive strength and backcalculated PCC modulus of elasticity were each approximately 10 percent less for distress concrete pavements than those obtained on the non-distressed Aggregate Test Road. Additional testing would need to be done to confirm whether this finding is statistically significant.
- Permeability of the concrete was assessed in the field using an air permeability test and in the laboratory using the rapid chloride permeability test. The non-distressed test sites (Test Site Nos. 0 through 4A) comprising the Aggregate Test Road all had RCPT results indicating low permeability. The general trend was increasing permeability based on RCPT results for distressed concrete, although no definitive trend was observed. This is in contrast to the air permeability results that found two of the non-distressed Aggregate Test Road sections (Test Site Nos. 1 and 4A) to be "permeable." What is interesting is both these sites are constructed with relatively porous coarse aggregate, Test Site No. 1 being slag and Test Site No. 4A being a porous dolomitic limestone. Further, the other test sites constructed with slag also had a "permeable" rating according to air permeability.
- The measured original air contents (ASTM C457) of concrete made with slag coarse aggregate were grossly higher than the specified values, ranging from 9.3 percent to 10.9 percent. This seemingly has had little effect on strength, but it indicates that difficulties in controlling the air content of concrete with slag coarse aggregate exist. Interestingly, one of the slag sites (Test Site No. 19) had the highest measured spacing factor of any of the test sites, even though the total original air content was 10.5 percent.
- Four of the test sites (Test Site Nos. 3, 19, 22, and 26A) had air-void system parameters that are considered marginal for protecting the paste from freeze-thaw damage. Although in no case was distress directly linked to paste freeze-thaw

deterioration, the entrained air void system is known to help alleviate pressures generated from other causes such as ASR and sulfate attack.

- Aggregate freeze-thaw deterioration was observed in six test sites (Test Site Nos. 12, 22, 26A, 26B, 27, and 28), all of which were constructed prior to 1975. The deterioration was primarily linked to large, carbonate aggregates from natural gravel sources, and to a lesser degree chert coarse aggregates. It is a testament to the effectiveness of MDOT's policies regarding aggregate screening through the use of MTM 115 that no aggregate freeze-thaw deterioration was observed in more recently constructed pavements
- Alkali-silica reactivity was observed in eight test sites (Test Site Nos. 4, 12, 19, 26A, 26B, 27, 28, and 29). In the case of the Clare Test Road (Test Site Nos. 26A, 26B, and 27), it was associated predominantly with various cherts, impure cherty carbonates, and sandstones coarse aggregate particles that are a constituent of the coarse aggregate. In Test Site Nos. 12 and 22, chert particles present in the natural gravel show signs of being deleteriously reactive, being linked both to alkali-silica reaction product and cracking. In the case of more recently constructed projects, the unique occurrence of ASR in the chert constituent of the fine aggregate in concrete having slag coarse aggregate is most notable. These same chert constituents, which are highly reactive and deleterious in the slag concrete, have been observed to be rather benign in concrete made with natural aggregates. Exactly why this is so is unknown, but enough evidence exists to suggest that a relationship exists between the ASR and the slag coarse aggregate.
- It has been observed that a unique deterioration mechanisms seem to be at work in distressed concrete pavements constructed with slag coarse aggregate (Test Sites Nos. 4, 19, and 29). The deterioration is manifest as a complete breakdown of the concrete matrix, generally in the vicinity of joints. Microscopically, there is strong evidence of calcium sulfide dissolution near the contact zone with the hydrated cement paste, a preponderance of calcium hydroxide in the hydrated cement paste, and secondary ettringite filling adjacent voids and cracks. In addition, sulfate extractions indicate excess sulfate is present in all concrete made with slag coarse aggregate. A hypothesis is emerging that the calcium sulfide that is in contact with the pore solution in the hydrated cement paste is undergoing an oxidation reaction resulting in dissolution. The calcium is precipitating as calcium hydroxide in the vicinity of the slag particles and the sulfate reacts with aluminate resulting in a type of internal sulfate attack.
- It has been observed that the concrete containing Class F fly ash (Test Site Nos. 0 through 4A) seems to have excellent durability properties. The RCPT results indicate "low" permeability and the chert constituents of the fine aggregate, although known to be reactive, are not involved in deleterious reactions, even in the presence of slag coarse aggregate. Further, although the concrete containing slag coarse aggregate (Test Site No. 1) has excess sulfates and a relatively high rate of ettringite infilling in the air voids, no damaging effect of sulfate attack is observed. It is known that Class F fly ash can help prevent ASR and sulfate attack, and the observations from this study support this conclusion. In contrast, the only test site constructed with Class C fly ash (Test Site No. 4) has undergone rapid deterioration as discussed previously.

RECOMMENDATIONS FOR FUTURE WORK

The following are recommendations for future work based on the results of this study:

- The hypothesis regarding the dissolution of calcium sulfide should be tested. The process is complex with multiple variables impacting the rate at which it might occur, or even if it occurs at all. Obviously, this process is not well enough understood at this time to even clearly implicate the calcium sulfide as being part of the observed deterioration, but enough evidence exist (elevated sulfate contents, observations of evacuated calcium sulfide dendrites, and the presence of considerable amounts of ettringite filling cracks and voids) to support a more in-depth analysis of this hypothesized phenomenon. It is recommended that a controlled laboratory study or studies be initiated to investigate the following:
  - The dissolution process and how it is effected by cement properties and total alkalinity.
  - The relationship between ASR in the chert constituent of the fine aggregate and the presence of slag coarse aggregate. This study must evaluate the effect of chert volume and alkalinity.
  - The ability of fly ash and GBFS to mitigate the effects of calcium sulfide dissolution and ASR in the fine aggregate. Both Class F and Class C fly ashes must be investigated.
- The observations regarding the volume and characteristics of air entrained in concrete containing slag coarse aggregate is interesting. It is possible that the coarseness of the slag particle may lead to more vigorous agitation of the fresh paste during mixing, resulting in increase air void formation. Also, the densified paste region characterized by unhydrated cement grains adjacent to the slag particles should be studied to determine its effect, if any, on the observed deterioration. For example, this might lead to unanticipated shrinkage, both physical and chemical, which in turn could produce cracking at the interface.
- The Aggregate Test Road should continue to be monitored. If signs of MRD emerge, an investigation should ensue to determine the cause of the distress.
- A parametric study of all slag concrete pavements should be conducted using mix design and construction data as well as field inspections. The purpose of this study is to determine if certain mixture design or construction variables have contributed to either a decrease or increase in durability. This limited study has found that Class F fly ash might offer a way to improve the durability of concrete made with slag coarse aggregates, whereas Class C fly ash has had an apparently negative impact. A more detailed large-scale study should be implemented to confirm this finding and determine if other variables are also instrumental.

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