Guidelines for Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements Volume 2: Guidelines Description and Use

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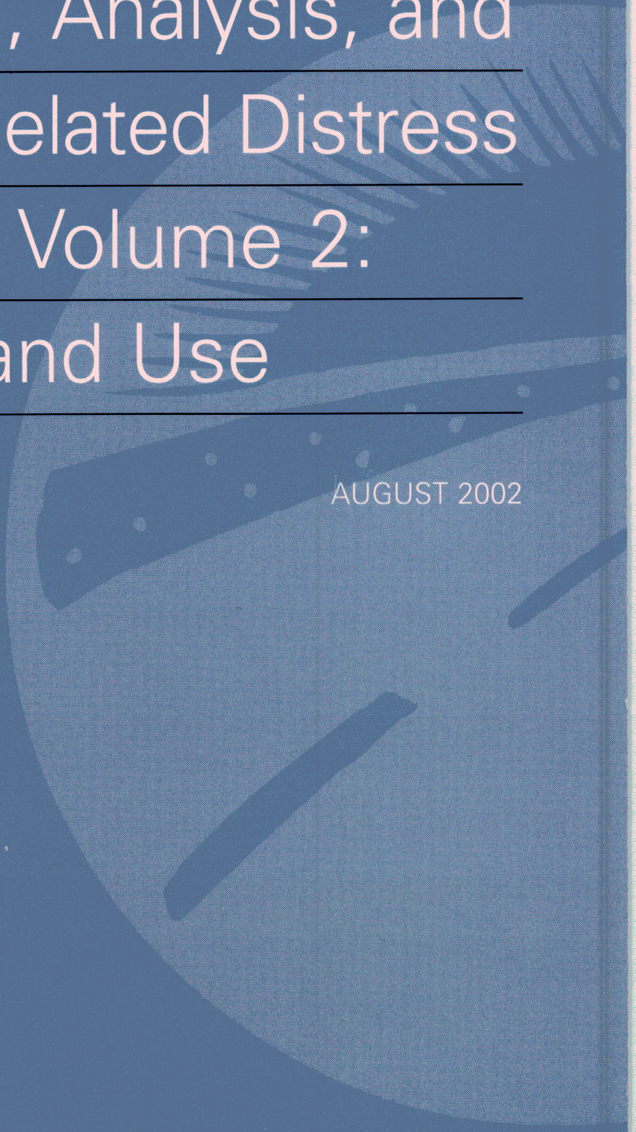

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Federal Highway Administration

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FOREWORD

This report presents a set of guidelines resulting from the investigation of materials-related distress (MRD) in concrete pavements. The performance of concrete pavements can be adversely affected by the concrete's inability to resist the environment in which it is placed. This lack of durability can occur even when the structural design of the pavement is adequate. The distresses that result from this lack of durability are referred to herein as MRD. The guidelines presented cover three major areas of MRD: field distress surveys, pavement sampling and sample handling; laboratory testing, data analysis and interpretation; and treatment, rehabilitation, and prevention. This report is Volume 2 of a three-volume set, and documents the guidelines developed and their use. Volume 1 is the Final Report, which documents the work conducted in the study. Volume 3 is the Case Studies Using the Guidelines, which provides examples of application of the guidelines developed.

These guidelines were developed with the practitioner in mind, and therefore this report will be of interest to those involved in evaluating concrete distress and performance, as well as those involved in the design and construction of concrete pavements. Sufficient copies are being distributed to provide 10 copies to each FHWA Resource Center, 5 copies to each FHWA Division, and 5 copies to each State highway agency. Direct distribution is being made to the FHWA Division Offices. Additional copies may be purchased from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Virginia 22161.

T. Paul Teng, P.E.
Director, Office of Infrastructure
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16. Abstract

The performance of concrete pavements can be adversely affected by the concrete's inability to resist the environment in which it is placed. This lack of durability can occur even when the structural design of the pavement is adequate. The distresses that result from this lack of durability are referred to herein as materials-related distress (MRD). This report documents the investigation of MRD in concrete pavements and the development of a set of systematic guidelines for the evaluation of MRD. The guidelines developed cover three major areas of MRD: field distress surveys, pavement sampling, and sample handling; laboratory testing, data analysis, and interpretation; and treatment, rehabilitation, and prevention.

This report is Volume 2 of a three-volume set, and documents the development and use of the guidelines developed. The other two reports are:

FHWA-RD-01- 163, Volume 1: Final Report
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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

TABLE OF CONTENTS

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

Section Page

TABLE OF CONTENTS (CONTINUED)

TABLE OF CONTENTS (CONTINUED)

LIST OF FIGURES

LIST OF FIGURES (CONTINUED)

LIST OF TABLES

GUIDELINE I — FIELD DISTRESS SURVEY, SAMPLING, AND SAMPLE HANDLING PROCEDURES FOR DISTRESSED CONCRETE PAVEMENTS

INTRODUCTION

As part of FHWA Project No. DTFH61-96-C-00073 entitled *Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements,* the following three guidelines have been developed to provide pavement engineers and field and laboratory personnel with a systematic procedure for the identification, evaluation, treatment, and prevention of materials-related distresses (MRD) in portland cement concrete (PCC) pavements:

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

These guidelines have been prepared with the practitioner in mind. Detailed background information used in the development of these guidelines can be found in the project Final Report.

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

Unfortunately, the complexity of the physical and chemical interactions involved in the development and manifestations of some types of MRD, as well as gaps in the current knowledge base, can often prevent absolute identification of the primary mechanism of distress. Also, it is not uncommon that more than one distress mechanism may be at work, a condition that further complicates the identification process. Nevertheless, it is expected that these systematic data collection and analysis processes will assist in building a knowledge base that does not currently exist, and which in turn can be used by future researchers to address questions that are currently unanswered.

MRD in PCC Pavements

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, 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 initial construction of the pavement.

Pavement distresses that are directly related to the interaction between the concrete materials used to construct the pavement and the environment are herein referred to as MRD. 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 of sufficient strength ensured durability, 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 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 to avoid durability problems.

Types of MRD

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 be attributed to either physical or chemical mechanisms, although the two types of mechanisms often act together to bring about the development of distress. Furthermore, MRD due to multiple causes may develop together, thereby complicating the determination of the exact cause(s) of material failure.

The MRD types considered in this series of guidelines are grouped as those caused by physical mechanisms and those cause by chemical mechanisms. Table I-1 summarizes these distresses, along with general information on their causes, appearance, and prevention; a description of these distresses is provided below.

Deterioration Due to Physical Mechanisms

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

Table I-1. Summary of key MRDs.

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/c* below 0.45, minimize tricalcium aluminate content in cement, use blended cements, use pozzolans.
Internal Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area.	Formation of ettringite from internal sources of sulfate that results in either expansive disruption in the paste phase or fills available air-voids, reducing freeze-thaw resistance.	1 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.

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 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. If adequate air-void space is available, all of the freezable water will eventually diffuse to the freezing sites inside the air voids, reaching a state of equilibrium. If the air-void space is inadequate, equilibrium cannot be reached, and osmotic pressures sufficient to fracture the paste can result.

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

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

Deicer scaling/deterioration is more likely to occur in concrete that has been over-vibrated or improperly finished, actions that create a weak layer of paste or mortar either at or just below the surface (Mindess and Young 1981). This weakened layer is more susceptible to hydraulic pressures. Even adequately air-entrained concrete can be susceptible to the development of salt scaling. Recommendations for the prevention of salt scaling include providing a minimum cement content of 335 kg/m^3, limiting the water–cement ratio (*w/c*) to a maximum of 0.45, providing adequate curing, and providing an absolute minimum of 30 days of environmental exposure before allowing the application of deicing chemicals [American Concrete Pavement Association (ACPA) 1992].

Freeze-Thaw Deterioration of Aggregate 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 susceptible aggregates are of sedimentary origin and are most commonly composed of limestone, dolomite, or chert (Stark 1976).

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

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

Deterioration Due to Chemical Mechanisms

Alkali–Silica Reactivity (ASR) is most commonly associated with undesirable chemical reactions between alkalis in the cement paste (commonly reported as percent Na_2O plus 0.658 x 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). ASR can also lead to internal horizontal cracks at greater depths within the slab. Upon continued expansion, joint spalling, blowups, shoving of fixed structures, and other pressure-related distresses in the pavement can occur. A handbook depicting ASR distress in pavements and highway structures is available to aid in its identification (Stark 1991).

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

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

The concrete pore solutions are primarily alkali hydroxide solutions of high concentration, which react readily with reactive forms of silica (Stark et al. 1993). As the aggregate reactivity increases, gel reaction products can be formed with lesser concentrations of alkali (Farny and Kosmatka 1997). As the alkalinity (and the pH) of the pore solution increases, the potential for alkali-silica reaction increases as even more stable forms of silica become susceptible to attack.

The presence of moisture allows migration of alkali ions to reaction sites and the resulting gel absorbs moisture, leading to expansion (Stark et al. 1993). Relative humidity levels above 80 percent indicate that moisture is sufficiently available for absorption by ASR gel, and recent research has shown that pavement concrete in all climates likely will be sufficiently damp to continuously support expansive ASR (Stark et al. 1993).

Other factors influencing the development of ASR include the total alkali content of the concrete, the presence of external sources of 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 ASR reaction process is provided by Helmuth (1993).

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

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

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

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

Alkali–Carbonate Reactivity (ACR) is another distress caused by an undesirable chemical reaction between the pore solution and aggregate; in this case, the reaction is between the alkalis in the pore solution and certain dolomitic carbonate aggregates containing a characteristic reactive texture of dolomite rhombs in a clayey fine-grained matrix. Although the mechanisms for ACR are not as well understood as those for ASR, it has been established that dedolomization occurs, that is, the decomposition of dolomite into calcium carbonate and magnesium hydroxide, which is accompanied by expansion. This expansion may be due to a combination of migration of alkali ions and water molecules into the restricted space of the fine-grained matrix surrounding the dolomite crystal, migration of these materials into the crystal, and the growth and arrangement of the dedolomitization products, especially brucite (Farny and Kosmatka 1997, ACI 1998). Other factors influencing the development of ACR include maximum size of the reactive aggregate (rate/degree of expansion decreases with decreasing aggregate size) and pore solution alkalinity (increasing pH levels 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 nonsusceptible aggregates or reducing the maximum size of susceptible aggregates can minimize the deleterious effect of the reaction (ACI 1998). Limiting the alkali content in the cement is another method employed to prevent ACR, but the alkali content must be lower than that typically used to prevent ASR (ACI 1998). However, unlike ASR, pozzolans are not effective in controlling alkali–carbonate reaction (Farny and Kosmatka 1997; ACI 1998).

External Sulfate Attack (ESA) can occur due to the penetration of external sulfate ions (present in groundwater, soil, deicing chemicals, etc.) into the concrete. Although the mechanism of sulfate attack is complex, it is primarily thought to be caused by two chemical reactions: 1) the formation of gypsum through the combination of sulfate and calcium ions, and 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/c* ratio 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 ($3CaO \bullet Al_2O_3 \bullet 3CaSO_4 \bullet 32H_2O$).

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.

It is believed that both SEF and DEF are forms of internal sulfate attack, but result for different reasons. SEF is commonly a product of concrete degradation, characterized by the dissolution and subsequent precipitation of ettringite into available void space and in pre-existing microcracks. SEF 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 (current 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 coarse aggregate particles of similar size along the interfacial zone.

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.

Corrosion of Embedded Steel 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.

Development of Advisory Guidelines

Need for Advisory Guidelines

These guidelines have been prepared to assist pavement engineers and field and laboratory personnel in the identification, evaluation, treatment, and prevention of MRD in concrete pavements. Investigations of concrete pavement deterioration have been conducted for years by numerous agencies, laboratories, and institutions, each with their own set of guidelines and testing procedures. Some State highway agencies (SHAs) have studied MRD in great detail, establishing unique testing protocols and material specifications. However, other agencies are not familiar with these procedures. In summary, there is a lack of information on the subject and, where information exists, there is little standardization regarding nomenclature, test methods, and identification procedures. The lack of standardization in test methods and data collection procedures makes it difficult to reach consensus on diagnoses and to analyze the performance of concrete pavements on a national basis.

Purpose and Objectives of Guidelines

The overall purpose of these guidelines is to provide a standardized approach for field evaluation, sampling and handling, and laboratory procedures for use in identifying MRD in pavements. Furthermore, these guidelines will provide useful information on effective measures for the economical preservation of PCC pavements identified as suffering MRD and guidance for the construction of long-lasting PCC pavements. It is expected that these guidelines will assist in the free flow of information among various agencies, laboratories, and institutions so that all benefit from the findings of each. Figure I-1 illustrates the flow of the three guidelines involved in the identification, treatment, and prevention of MRD.

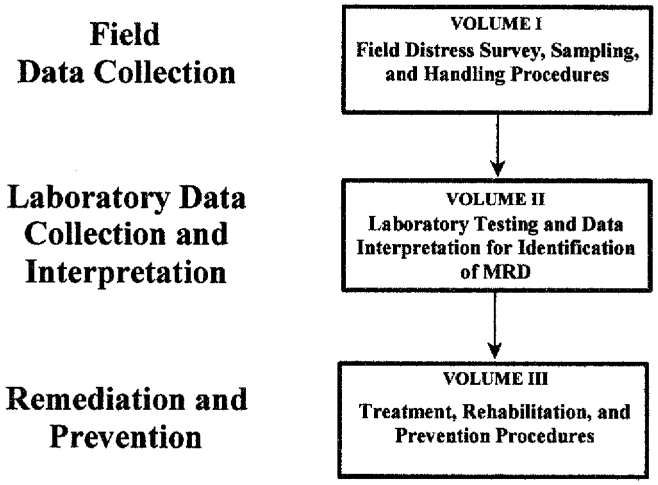


Figure I-1. Guidelines for identification, treatment, and prevention of MRD.

FIELD DISTRESS SURVEY PROCEDURES

A standardized, systematic, and comprehensive field data collection effort is required on PCC pavements exhibiting MRD in order to aid in the overall identification of the distress mechanisms. A variety of activities are involved in this effort, including the collection of historical pavement design and construction information, the selection of representative pavement inspection sites, the conduct of the actual distress surveys, the characterization of the MRD, and the photographic documentation of the pavement. This overall process is depicted in figure I-2.

This section of this guideline presents information on the conduct of distress surveys and associated field testing of concrete pavements that are suspected of exhibiting MRD. It presents a step-by-step approach for conducting distress surveys of such pavements (largely following the SHRP Long-Term Pavement Performance [LTPP] survey procedures) as well as techniques and field procedures for characterizing the various MRD types.

Data Collection Procedures

The following sections describe the approach for the various field data collection activities to be conducted as part of the pavement field evaluation. These are intended primarily as a guide for assisting in examining and identifying the manifestations of MRD in an existing PCC pavement, but if conducted at regular intervals during the pavement's service life, they are useful in establishing the rate of deterioration. The results of this survey will be used to determine if the observed distress is the likely result of an MRD and whether coring and laboratory analysis is justified. The field survey process is summarized in table I-2.

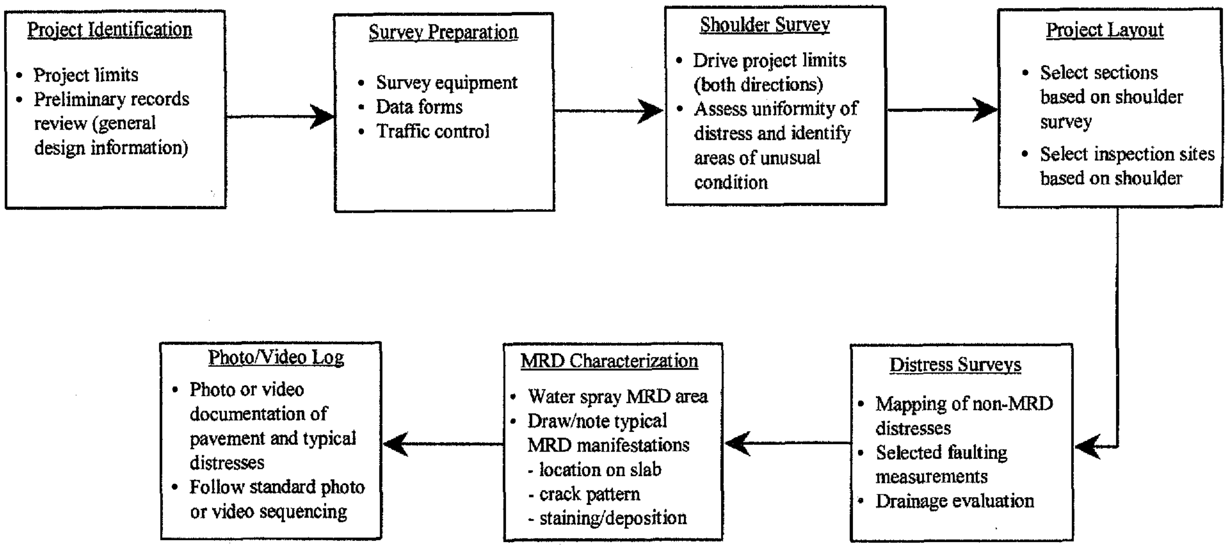


Figure I-2. Flow of field data collection activities.

Table I-2. Summary of distress survey components.

Field Testing Component	Purpose
Project Identification and Records Review	• Identify project location • Identify basic design, construction, and materials information
Survey Preparation	• Assemble field survey equipment • Assemble data collection forms • Arrange for traffic control
Shoulder Survey	• Assess overall pavement condition, uniformity of distress, topography, and traffic volumes • Identify location of bridges, ramps, curves, intersections, etc., that could affect the selection of section
Project Layout	• Select inspection site(s) for inspection • Use 150-m-long inspection sites • Additional inspection sites may be needed depending on condition
Pavement Distress Surveys	• Establish overall pavement performance
MRD Characterization	• Characterize existing MRD in terms of its cracking pattern, staining, exudate, scaling, and presence of vibrator trails
Photo/Video Documentation	• Provide photographic/videotape documentation of pavement condition

Project Identification and Records Review

Prior to any fieldwork, a review of the design and construction records must be conducted on the project to be evaluated. The general information sought is needed to identify the limits of the project, and to establish some basic climatic information that is important in determining what MRD might be at work. Structural design, shoulder, and foundation information are needed to assist in the distress survey mapping and evaluation.

This information must be collected for each project being evaluated. A project is defined as that length of pavement over which the structural design, date and method of construction, and paving and foundation materials are the same. This helps to ensure that the causes of distress can be more reliably determined. If significant changes in design or materials occurs (for example, if a different type of aggregate source was used), then these areas should be broken out as separate projects for evaluation. If considerable differences in the type or quantity of distress are noted within a given project, the project should be divided into different sections, each of which is subsequently evaluated. If the distress is more or less uniformly distributed over the entire project, only a single section will be evaluated. Figure I-3 presents a form that can be used to record this information. A reproducible copy of this form, like all field data collection forms discussed in this guideline, is provided in appendix A of this guideline.

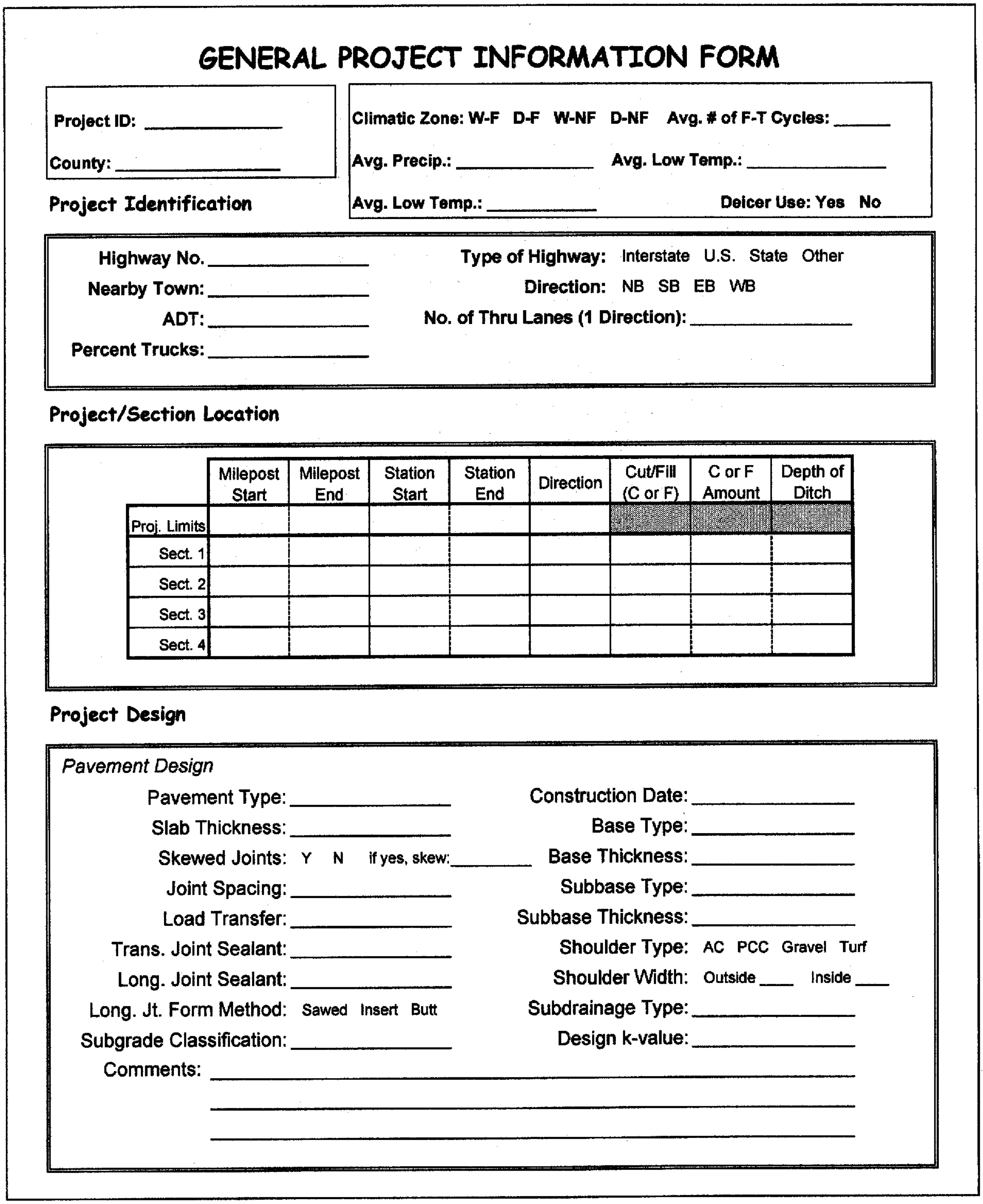
The PCC mix design information is useful in assessing the factors that contributed to the MRD and in selecting locations for sampling and testing. Figure I-4 can be used to record this information. Relevant properties of the fine and coarse aggregate are recorded in figures I-5 and I-6, respectively. Finally, relevant construction information can be recorded in figure I-7. By obtaining these data prior to any fieldwork, an effective and efficient field-testing program can be developed.

Survey Preparation

In order to ensure that the field surveys proceed smoothly and efficiently, adequate planning and preparation must be made for their conduct. This planning and preparation includes assembling the necessary equipment, preparing the data collection forms, and organizing the required traffic control arrangements. These items are discussed below.

Field Survey Equipment

A checklist of basic equipment required for the field surveys is provided in table I-3. Additional equipment may be needed if more intensive field data collection will be conducted.



GENERAL PROJECT INFORMATION FORM

Project ID: ______________

County: ________________

Project Identification

Climatic Zone: W-F D-F W-NF D-NF **Avg. # of F-T Cycles:** _____

Avg. Precip.: ______________ **Avg. Low Temp.:** ______________

Avg. Low Temp.: ___________ **Deicer Use:** Yes No

Highway No. ________________

Nearby Town: ________________

ADT: ________________

Percent Trucks: ________________

Type of Highway: Interstate U.S. State Other

Direction: NB SB EB WB

No. of Thru Lanes (1 Direction): ________________

Project/Section Location

	Milepost Start	Milepost End	Station Start	Station End	Direction	Cut/Fill (C or F)	C or F Amount	Depth of Ditch
Proj. Limits						▨	▨	▨
Sect. 1								
Sect. 2								
Sect. 3								
Sect. 4								

Project Design

Pavement Design

Pavement Type: ________________

Slab Thickness: ________________

Skewed Joints: Y N if yes, skew: ________

Joint Spacing: ________________

Load Transfer: ________________

Trans. Joint Sealant: ________________

Long. Joint Sealant: ________________

Long. Jt. Form Method: Sawed Insert Butt

Subgrade Classification: ________________

Construction Date: ________________

Base Type: ________________

Base Thickness: ________________

Subbase Type: ________________

Subbase Thickness: ________________

Shoulder Type: AC PCC Gravel Turf

Shoulder Width: Outside ____ Inside ____

Subdrainage Type: ________________

Design k-value: ________________

Comments: ____

Figure I-3. General project information form.

Table I-3. Field survey equipment checklist.

Item	✓
Measuring Wheel	
Marking Paint	
Faultmeter	
Ruler/Scale	
Pocket Knife	
Clipboard/Pencils/Survey Forms	
Water Spray Bottle	
Magnifying Glass	
Camera or Camcorder (with extra film/tapes)	
Flashing Warning Light	
Safety Vests and Hard Hats	
LTPP Distress Identification Manual	
Project Location	
Local Contact Name/Number	

Mix Design Information

Project ID: ____________ **Dates of Construction:** ____/____/____ to ____/____/____

Section: ____________

Cementitious Materials

Cement Type: ____________ **Source:** ____________

Total Alkalis (%): ____________ (Na equiv.) **Fineness:** ____________

Mill Certification?: Yes No If "Yes", Attach mill certification to this form.

Fly Ash Type: ____________ **Source:** ____________

Total Alkalis (%): ____________ (Na equiv.) **Fineness:** ____________

Certification?: Yes No ____________ If "Yes", Attach certification to this form.

Other Finely Divided Materials?: Yes No If "Yes", describe and attach documentation.

____

PCC Mixture Design

Design Weight Proportions (kg/cubic meter or pounds/cubic yard)

Cement: ____________ **Other Cementitious:** ____________

Fly Ash: ____________ **Fine Aggregate:** ____________

Water: ____________ **Coarse Aggregate:** ____________

Unit Weight: ____________ **Percent Air:** ____________

***w/c*:** ____________ **Compressive Strength:** ________ @ ________ days

Slump: ____________ **Other Standard Tests:** ____________

____

Admixtures

Air Entrainment

Source: ____________ Type: ____________ Rate: ____________

Admixture 2

Source: ____________ Type: ____________ Rate: ____________

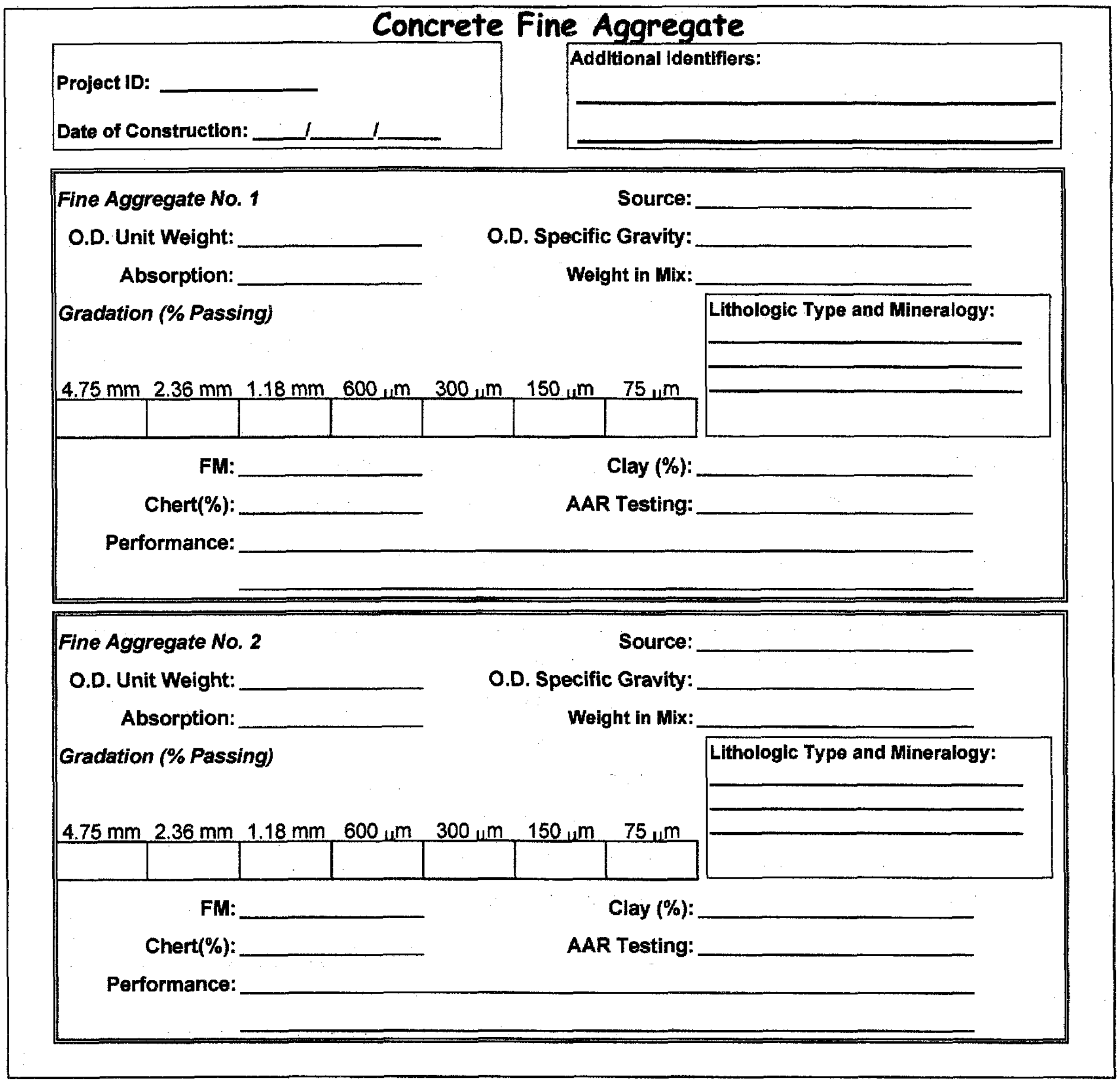
Admixture 3

Source: ____________ Type: ____________ Rate: ____________

Comments: ____

____

Figure I-4. Concrete mixture design information form.



Concrete Fine Aggregate

Project ID: ____________

Date of Construction: ____/____/____

Additional Identifiers:

____

____

Fine Aggregate No. 1

Source: ____________

O.D. Unit Weight: ____________

O.D. Specific Gravity: ____________

Absorption: ____________

Weight in Mix: ____________

Gradation (% Passing)

4.75 mm	2.36 mm	1.18 mm	600 μm	300 μm	150 μm	75 μm

Lithologic Type and Mineralogy:

____

____

____

FM: ____________

Clay (%): ____________

Chert(%): ____________

AAR Testing: ____________

Performance: ______________________________

____

Fine Aggregate No. 2

Source: ____________

O.D. Unit Weight: ____________

O.D. Specific Gravity: ____________

Absorption: ____________

Weight in Mix: ____________

Gradation (% Passing)

4.75 mm	2.36 mm	1.18 mm	600 μm	300 μm	150 μm	75 μm

Lithologic Type and Mineralogy:

____

____

____

FM: ____________

Clay (%): ____________

Chert(%): ____________

AAR Testing: ____________

Performance: ______________________________

____

Figure I-5. Concrete fine aggregate information form.

Concrete Coarse Aggregate

Project ID: ______________

Date of Construction: ____/____/____

Additional Identifiers: ______________________________

____

Coarse Aggregate No. 1

Source: ______________

O.D. Unit Weight: ______________

O.D. Specific Gravity: ______________

Absorption: ______________

Weight in Mix: ______________

Gradation (% Passing)

63 mm	50 mm	37.5 mm	25 mm	19 mm	12.5 mm	9.5 mm

4.75 mm	2.36 mm	1.18 mm	600 μm	300 μm	150 μm	75 μm

Lithologic Type and Mineralogy: ______________

____

____

LA Abrasion: ______________

Clay Lumps (%): ______________

Chert(%): ______________

Soundness: ______________ (Na or Mg)

F-T Durability: ______________

AAR Testing: ______________

Performance: ______________________________

____

Coarse Aggregate No. 2

Source: ______________

O.D. Unit Weight: ______________

O.D. Specific Gravity: ______________

Absorption: ______________

Weight in Mix: ______________

Gradation (% Passing)

63 mm	50 mm	37.5 mm	25 mm	19 mm	12.5 mm	9.5 mm

4.75 mm	2.36 mm	1.18 mm	600 μm	300 μm	150 μm	75 μm

Lithologic Type and Mineralogy: ______________

____

____

LA Abrasion: ______________

Clay Lumps (%): ______________

Chert(%): ______________

Soundness: ______________

F-T Durability: ______________

AAR Testing: ______________

Performance: ______________________________

____

Figure I-6. Concrete coarse aggregate information form.

Construction Information

Project ID: ______________ **Dates of Construction:** ____/____/____ to ____/____/____

Section: ______________

Mixing Plant

Type: ______________ **Make:** ______________

Capability: ______________ **Mixing Cycle:** ______________

Comments: ______________

____

Construction

Type of Truck: **Mixing in Transit?:** Yes No

Haul Time: ______________ **Curing Method:** ______________

Comments: ______________

Mix Properties During Construction

Range and Avg. Slump:______________ Range and Avg. Air:______________

Range and Avg. Strength:______________ Type:______________

PCC Mixture Temperature at Placement

Average:______________ Date:______________

Low:______________ Date:______________

High:______________ Date:______________

______________ ______________

Comments: ______________

Climatic Conditions During Paving

High Air Temperature:______________ Date:______________

Low Air Temperature:______________ Date:______________

Average Air Temperature:______________

Comments on Humidity: ______________

Comments on Wind: ______________

Does there appear to be a relationship between climatic conditions and observed distress? If so, prepare a plot of mixture temperatures along the project length to see if a correlation exists.

Figure I-7. Construction information form.

Data Collection Forms

Several different data collection forms are needed to conduct the pavement distress surveys. The first data collection form is the field distress survey form, illustrated in figure I-8. This form is adapted from the form used in the data collection activities of the LTPP program (SHRP 1993). All pavement distresses are to be recorded on this form, in addition to the *general area of occurrence* of the MRD on the pavement. The purpose of this form is to document the overall performance of the pavement, which may be helpful in the diagnosis of the MRD type and mechanisms. The overall pavement distress data are tabulated later on the distress summary form shown in figure I-9, which again is adopted from the LTPP program (SHRP 1993).

Two forms are used in the field data collection activities to characterize the MRD observed on the pavement surface. Figure I-10 is used to draw in detail the typical MRD occurring on a single slab (or several forms can be filled out to depict the range of conditions), including the crack pattern and the presence of staining and any exudation. Figure I-11 is then used to categorize the MRD in terms of its cracking pattern (location, extent, size), staining (location, color), material deposits or exudation (presence, color, extent), spalling, and scaling.

For each project evaluation, different numbers of each form are required:

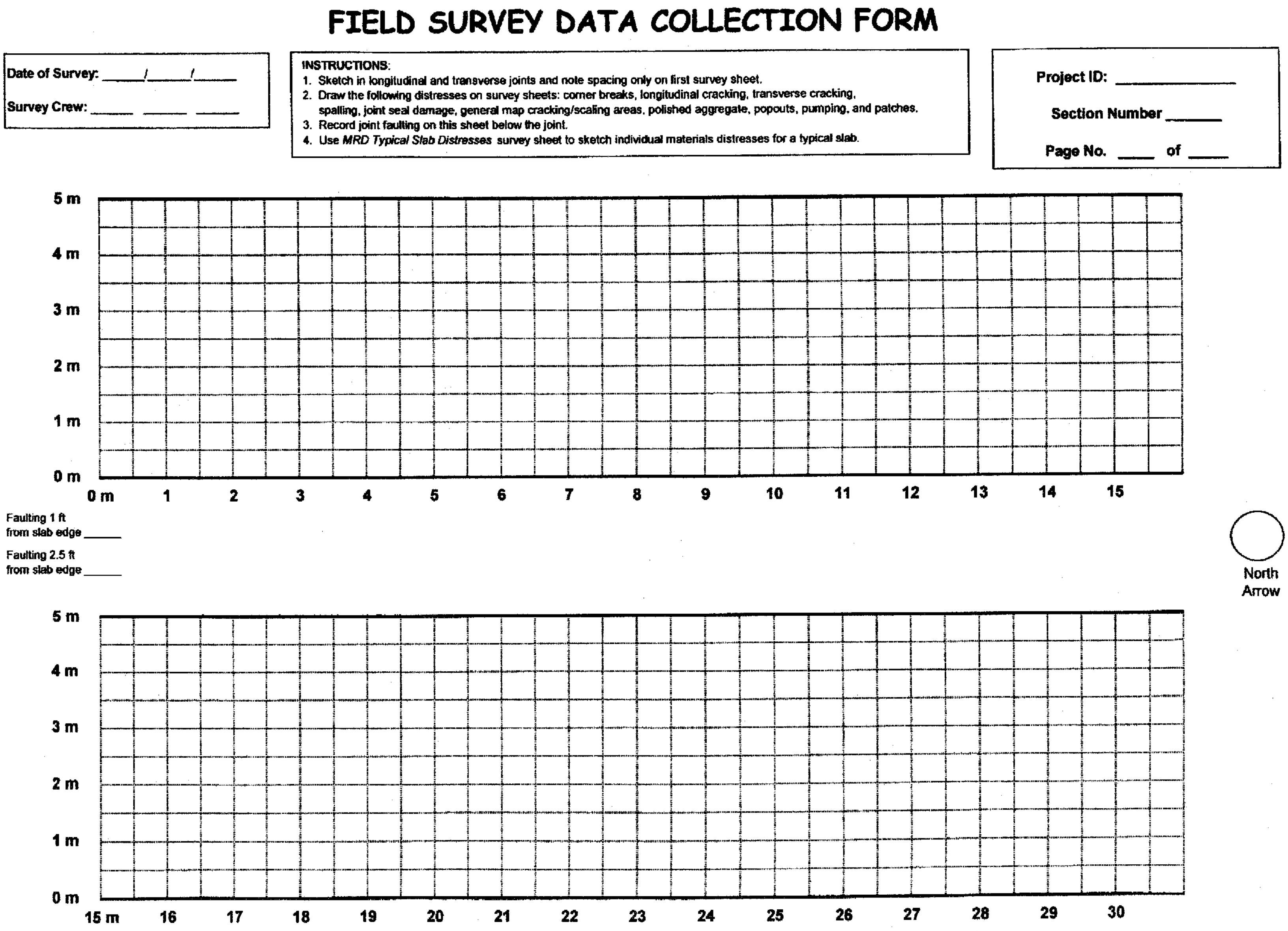
- Overall Pavement Distress Survey Form (figure I-8)—5 copies (for a total section length of 150 m, consistent with the monitoring work being done under the LTPP program).
- Overall Pavement Distress Summary Form (figure I-9)—1 copy.
- Typical MRD Survey Form (figure I-10)—1 to 2 copies (depending on range of conditions).
- MRD Characterization Form (figure I-11)—1 to 2 copies (depending on range of conditions).

Organization and Coordination of Field Survey Operations

A planning meeting should be held by the survey team, the traffic control crew, and representatives of the local highway agency to discuss the scope of work, general locations of the pavement test sections, schedules, duties and responsibilities, permits (if any), safety regulations and requirements, legal requirements, emergency actions, and other concerns (SHRP 1990). Requirements for traffic control, permits, insurance and other legal requirements related to field work vary between governing agencies, so the exact requirements should be determined early on. Names, addresses, and phone numbers of appropriate personnel should be exchanged to facilitate future communication.

Traffic Control Arrangements

The provision of adequate traffic control is extremely important for the safe conduct of the field surveys. Although some of the work can be performed from the shoulder, the pavement evaluator must be allowed on the pavement with the freedom to closely inspect the entire pavement surface. In addition, later sampling and testing activities will require complete access to the pavement; additional information on traffic control requirements for field sampling is described later in this guideline.



FIELD SURVEY DATA COLLECTION FORM

Date of Survey: _____/_____/_____

Survey Crew: _____ _____ _____

INSTRUCTIONS:
1. Sketch in longitudinal and transverse joints and note spacing only on first survey sheet.
2. Draw the following distresses on survey sheets: corner breaks, longitudinal cracking, transverse cracking, spalling, joint seal damage, general map cracking/scaling areas, polished aggregate, popouts, pumping, and patches.
3. Record joint faulting on this sheet below the joint.
4. Use *MRD Typical Slab Distresses* survey sheet to sketch individual materials distresses for a typical slab.

Project ID: ____________

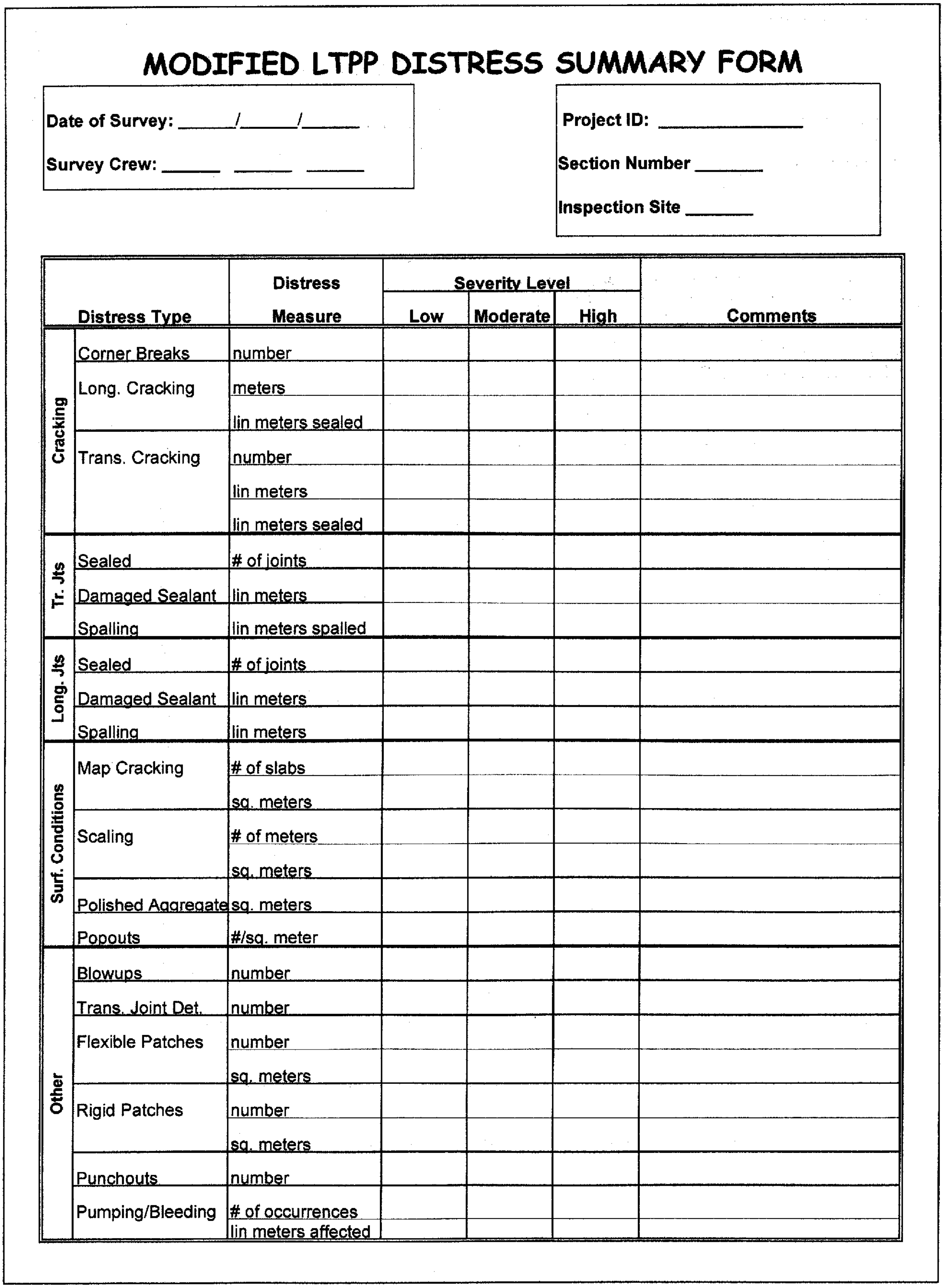
Section Number ______

Page No. ____ of ____

Faulting 1 ft from slab edge _____

Faulting 2.5 ft from slab edge _____

Figure I-8. Modified LTPP distress survey form (adapted from SHRP 1993).



MODIFIED LTPP DISTRESS SUMMARY FORM

Date of Survey: ______/______/______

Survey Crew: ______ ______ ______

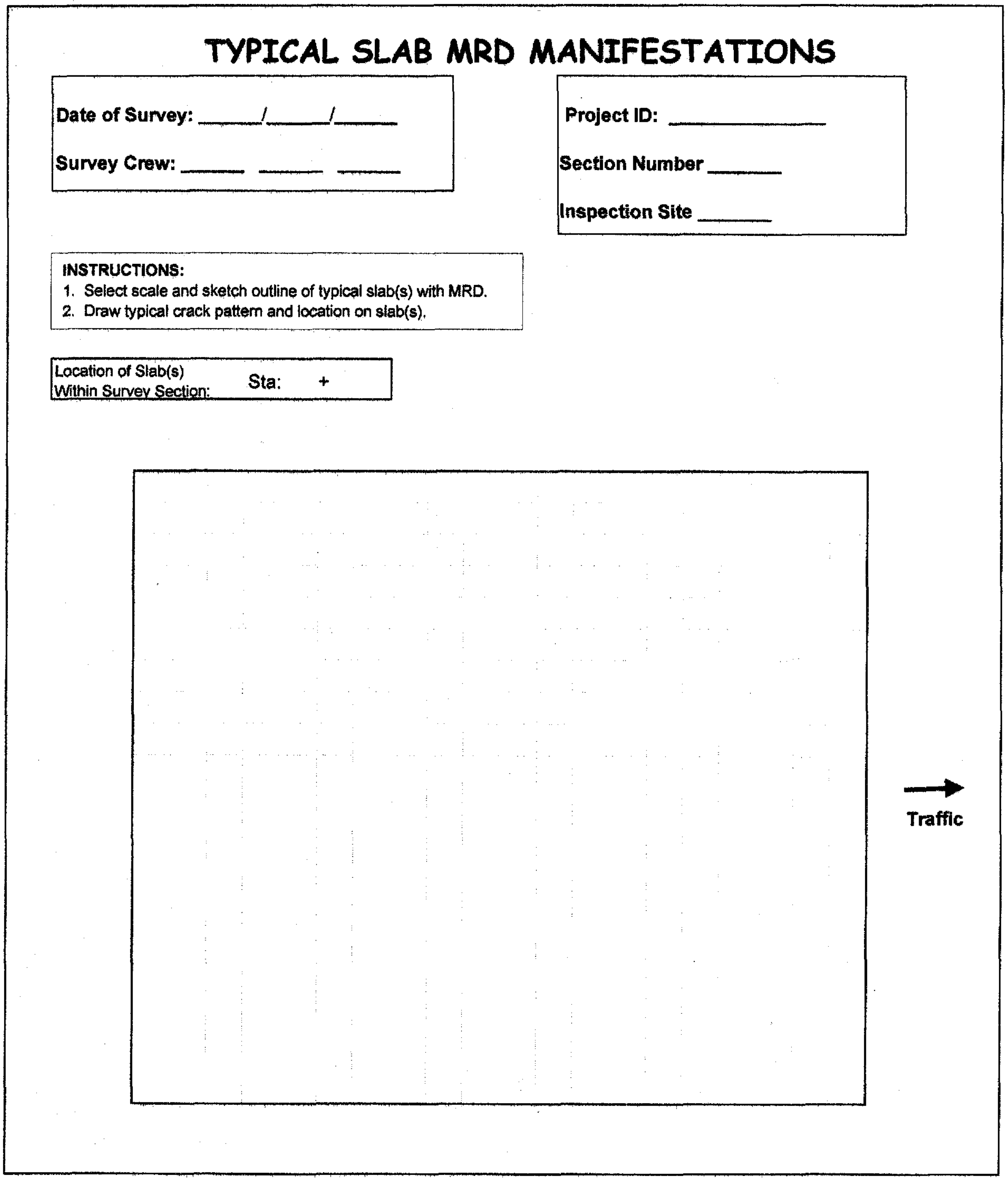
Project ID: ______________

Section Number _______

Inspection Site _______

	Distress Type	Distress Measure	Severity Level			Comments
			Low	Moderate	High	
Cracking	Corner Breaks	number				
	Long. Cracking	meters				
		lin meters sealed				
	Trans. Cracking	number				
		lin meters				
		lin meters sealed				
Tr. Jts	Sealed	# of joints				
	Damaged Sealant	lin meters				
	Spalling	lin meters spalled				
Long. Jts	Sealed	# of joints				
	Damaged Sealant	lin meters				
	Spalling	lin meters				
Surf. Conditions	Map Cracking	# of slabs				
		sq. meters				
	Scaling	# of meters				
		sq. meters				
	Polished Aggregate	sq. meters				
	Popouts	#/sq. meter				
Other	Blowups	number				
	Trans. Joint Det.	number				
	Flexible Patches	number				
		sq. meters				
	Rigid Patches	number				
		sq. meters				
	Punchouts	number				
	Pumping/Bleeding	# of occurrences				
		lin meters affected				

Figure I-9. Modified LTPP distress summary form.



TYPICAL SLAB MRD MANIFESTATIONS

Date of Survey: _____/_____/_____

Survey Crew: _____ _____ _____

Project ID: ____________

Section Number _____

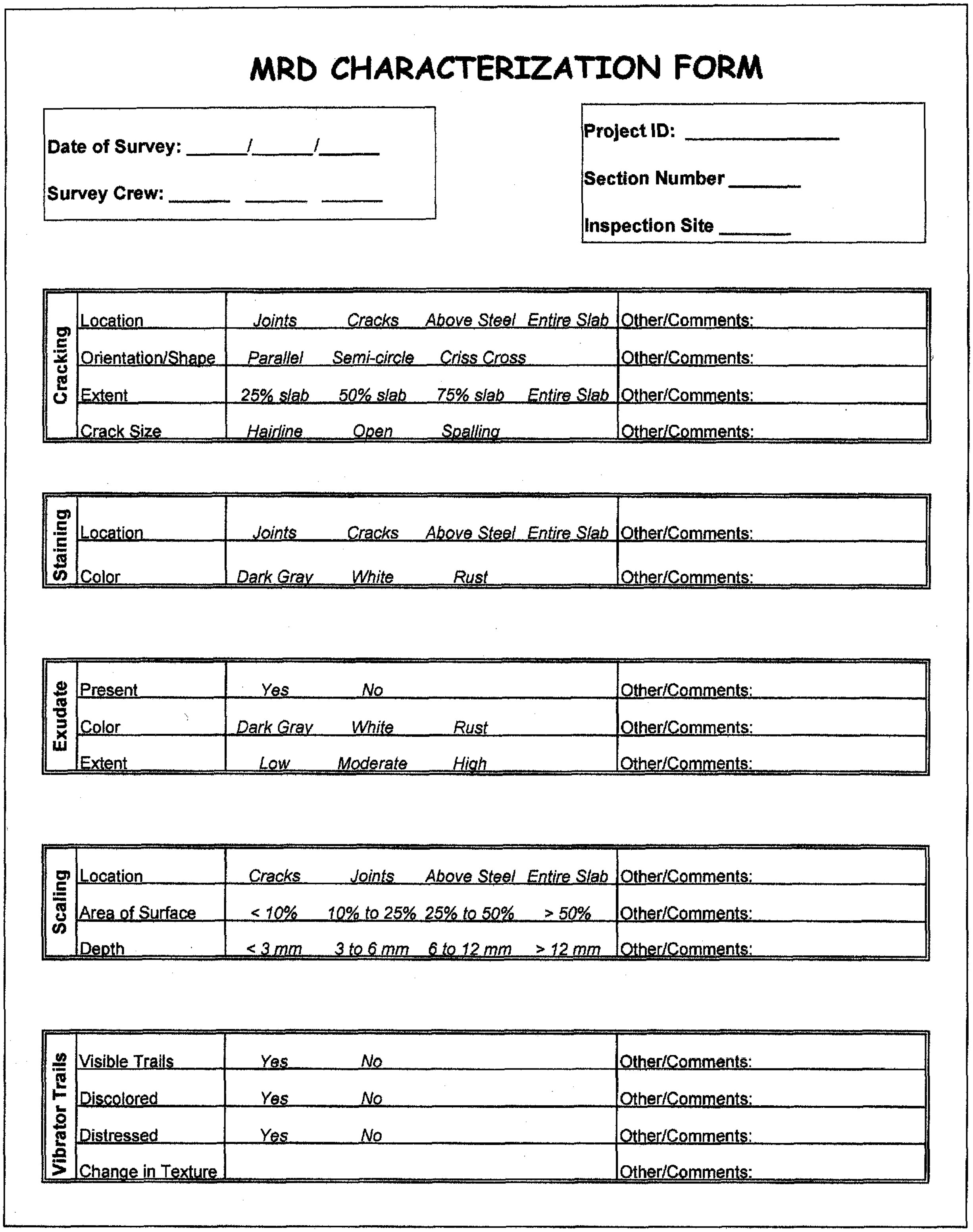
Inspection Site _____

INSTRUCTIONS:

1. Select scale and sketch outline of typical slab(s) with MRD.
2. Draw typical crack pattern and location on slab(s).

Location of Slab(s) Within Survey Section: Sta: +

Figure I-10. Survey form for illustrating typical MRD.



MRD CHARACTERIZATION FORM

Date of Survey: ______/______/______

Survey Crew: ______ ______ ______

Project ID: _______________

Section Number _______

Inspection Site _______

Cracking	Location	*Joints*	*Cracks*	*Above Steel*	*Entire Slab*	Other/Comments:
	Orientation/Shape	*Parallel*	*Semi-circle*	*Criss Cross*		Other/Comments:
	Extent	*25% slab*	*50% slab*	*75% slab*	*Entire Slab*	Other/Comments:
	Crack Size	*Hairline*	*Open*	*Spalling*		Other/Comments:

Staining	Location	*Joints*	*Cracks*	*Above Steel*	*Entire Slab*	Other/Comments:
	Color	*Dark Gray*	*White*	*Rust*		Other/Comments:

Exudate	Present	*Yes*	*No*			Other/Comments:
	Color	*Dark Gray*	*White*	*Rust*		Other/Comments:
	Extent	*Low*	*Moderate*	*High*		Other/Comments:

Scaling	Location	*Cracks*	*Joints*	*Above Steel*	*Entire Slab*	Other/Comments:
	Area of Surface	*< 10%*	*10% to 25%*	*25% to 50%*	*> 50%*	Other/Comments:
	Depth	*< 3 mm*	*3 to 6 mm*	*6 to 12 mm*	*> 12 mm*	Other/Comments:

Vibrator Trails	Visible Trails	*Yes*	*No*			Other/Comments:
	Discolored	*Yes*	*No*			Other/Comments:
	Distressed	*Yes*	*No*			Other/Comments:
	Change in Texture					Other/Comments:

Figure I-11. MRD characterization form.

The pavement survey work will require only one lane being closed, and on multilane facilities this is generally the outer traffic lane. However, if significant differences are observed between lanes, consideration may be given to evaluating multiple lanes. The direction to be evaluated will need to be selected for both multilane and two-lane roadways. If deemed important, both directions can be evaluated. All traffic control arrangements should be scheduled as far in advance as possible and should adhere to the guidelines provided in the *Manual on Uniform Traffic Control Devices* (MUTCD) [Federal Highway Administration (FHWA) 1993] or the agency's governing requirements.

Shoulder Survey

Prior to the set up of the traffic control and the conduct of the pavement distress surveys, it is useful to first drive the limits of the project to assess the overall pavement condition and the uniformity of distress levels, topography, and traffic volumes. This can be done by a crew of two while driving along the shoulder at slow speeds (8 to 16 km/h), noting general pavement conditions by reference post or by distance as measured from the beginning of the project with the vehicle's odometer. In addition, any prevailing roadway characteristics that might have an effect on the performance of the pavement (e.g., areas of cuts/fills, topography, traffic volume/loading) should be noted.

Shoulder surveys should be conducted in both directions when both directions represent the same project (structural design, mix design, construction, and so on). Any directional differences in performance or distress manifestations should be noted by location to assist later in dividing the project into multiple sections, if needed. If the distress is more or less uniformly distributed over the entire project, only a single section will be evaluated.

Other features that should be noted during the shoulder surveys include the location of bridges, on/off ramps, sharp vertical or horizontal curves, and any other factors that might affect the safe conduct of a pavement survey at that location.

Project Layout

After the shoulder survey has been conducted, the project must be divided into sections if distinctly different distress manifestations are evident. This is unnecessary if the distress is more or less uniformly distributed over the entire project. Within each section, one or more inspection sites are selected for field surveys and sampling. Inspection sites are short segments (typically 150 m in length) that are selected for the conduct of the field studies. As such, they are intended to be representative of the overall section, which may be several kilometers long.

Selecting Inspection Sites

The selection of appropriate inspection sites is an important part of the field survey investigation. It is desirable that the selected inspection sites reflect the typical conditions of the section and that they are fully accessible so that the required fieldwork can be conducted.

A great deal of information is expected to become available from the evaluation of the inspection sites, but because of the time and effort involved, it is desirable to minimize the number of sites while still maintaining an adequate representation of the project. Therefore, the number and location of inspection sites will depend largely on the consistency of the project. Generally, if the shoulder survey indicates that the extent and severity of MRD are fairly uniform throughout the length of the project, then a single inspection site is selected at a representative location for extensive testing and evaluation. If, however, the shoulder survey indicates a range of MRD severity levels, or areas where there is no distress and other areas where there is severe distress, the project should be divided into multiple sections if possible. If the MRD varies in such a way that it is impossible to identify distinct sections, multiple inspection sites should be selected that represent the variability observed. This will assist in determining why one area is performing differently than another. Possible reasons for differences in performance might be:

- Unknown documented differences in constituent materials. For example, aggregate type or source may have been changed over the length of a project, even though the project records indicate the same aggregate type/source was used over the entire length.
- Differences in drainage patterns, depth to water table, and topography. For example, areas of poor drainage may exhibit more severe distress than other areas as the presence of free moisture greatly accelerates the development of MRD.
- Differences in traffic levels or loadings. Although MRDs develop independently of traffic, higher traffic levels may exacerbate the appearance of the distress.
- Differences in application rates of deicers. For example, some portions of roadways (ramps, curves, hills, or other areas prone to icing) may receive greater applications of deicing materials, and this could accelerate the development of MRD at those locations.

The possibility of any of the above factors affecting the performance of the pavement should be considered and noted as the various inspection sites are evaluated.

During the selection of the inspection site to be surveyed, it is desirable to avoid areas that are unrepresentative of the overall conditions, and also areas that might compromise the safety of the survey crew. For example, an area exhibiting extensive transverse cracking because of settlement of an underlying culvert is not considered representative of the MRD or the performance of the entire project. Other areas to avoid are those:

- Near bridges or other structures because of their effect on the performance of the pavement.
- Near exit or entrance ramps due to the complexities of establishing and maintaining traffic control and the dangers of exiting/merging traffic.
- Near sharp horizontal or vertical curves where limited sight distances may compromise the safety of the field operations.

Inspection Site Length

To ensure consistency with the LTPP program, a nominal inspection site length of 150 m is recommended. This provides a sufficiently long site to provide reasonable insight into the performance of the pavement while maintaining some degree of "conciseness" to the field operations. However, the actual inspection site length may need to be adjusted so that entire slab lengths are included. For example, if the joint spacing of a project is 5 m, then the inspection site will consist of 30 (150/5) slabs. On the other hand, if the joint spacing is 4.5 m, then the inspection site will consist of 33.3 slabs (150/4.5), which is rounded up to 34 slabs. In this case, the actual inspection site length will be 153 m. The number of slabs is always rounded up to ensure that a minimum length of 150 m is achieved.

Additional Inspection Sites

Occasionally, there may be a need to include additional inspection sites as part of the evaluation of a project. This may occur for a number of reasons, including:

- Identification of areas within the section exhibiting significantly different performance than the rest of the project.
- Observation of an adjacent section of similar design that is exhibiting unique distress manifestations, or significantly less distress than the subject project.
- Encounter of an unexpected finding or development on a inspection site that requires additional verification or confirmation.
- Belief during or after the evaluation of an inspection site that it is not necessarily reflective of the actual conditions of the entire project.

Despite the identification of appropriate inspection sites and the possible evaluation of additional sites, it is still possible that specific follow-up surveys or field testing may be needed as information comes back from the results of the laboratory testing. This underscores the ability to return to the correct location of the inspection sites so that the follow-up tests can be accurately tied to the actual field location.

Pavement Distress Surveys

The conduct of the pavement distress surveys is a critical aspect of the overall evaluation of the project. Prior to the conduct of any pavement surveys, it is expected that suitable traffic control has been established in accordance with local guidelines so that the entire inspection site can be freely accessed. The distress survey to be conducted shall be performed in general accordance with the procedures and distress definitions found in the SHRP LTPP *Distress Identification Manual for the Long-Term Pavement Performance Project* (SHRP 1993).

It should be recognized that the purpose of conducting an LTPP-type distress survey is to provide a broad overview of the condition of the pavement by recording the presence and severity of such distresses as linear cracking, spalling, and faulting. However, it is not required to collect faulting for most MRD surveys. In addition, it is not the intent of an MRD survey to draw every crack associated with the distress acting on a pavement. Such documentation is time consuming and does not necessarily contribute to the identification of the distress. Instead, the general area of MRD will be noted on the LTPP forms and its physical manifestations characterized later on the MRD survey form.

Overview of Distress Surveys

This section provides a brief description of the LTPP pavement distress surveys that are to be conducted. Critical aspects of the distress survey procedures are described, with detailed information on conducting a pavement distress survey found in the *Distress Identification Manual* (SHRP 1993).

The survey process first begins with laying out the inspection site. This is accomplished by selecting the beginning point of the site (usually right at a reference post, station number, or other permanent marker or fixture to facilitate locating the inspection site in the future) and marking with paint its beginning at the transverse joint nearest the marker. The beginning point should be noted with a line and with a station number of "0+00"; by defining this starting location as station 0+00, all stationing along the inspection site can then be made relative to the beginning point.

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

After the section is laid out, the distress survey proceeds from the beginning of the site. All distresses occurring within the inspection site are noted and, where appropriate, drawn approximately to scale using the modified LTPP distress survey form (figure I-8).

To facilitate the recording of the various distresses, the distress map symbols shown in figure I-12 [for jointed concrete pavements (JCP)] and figure I-13 [for continuously reinforced concrete pavements (CRCP)] are provided. These are modified from the LTPP *Distress Identification Manual*, as attempts have been made to focus on key distress types and also to reduce the distress survey to a *recording* of pavement distress as opposed to a *diagnosis* of pavement distress. The most significant modification is that the LTPP *Distress Identification Manual* includes D-cracking as a distress, but this characterization assumes a field diagnosis of an MRD, which is in conflict with the goals and objectives of these guidelines. Therefore, for the purposes of this procedure, D-cracking has been removed as a distress type, but is instead recorded based on its cracking and staining pattern. The primary distresses that are to be evaluated during the distress surveys are listed in table I-4.

Again, it is noted that the many individual cracks associated with MRD are not drawn in on the LTPP survey distress form, but rather the approximate areas or limits of the MRD are recorded. If the MRD occurs over an entire slab or an entire area, then this is noted in the margin of the distress form.

It is generally most useful to have two raters involved in conducting the pavement distress surveys. One serves as the recorder and is responsible for completing the data forms, including the drawing and recording of all pavement distresses. The second rater is responsible for obtaining measurements where needed (e.g., joint faulting, joint widths, areas, and so on) and also to help in detecting the various pavement distresses.

The primary distress survey will generally be conducted in only one lane (e.g., the outer lane of a multilane facility or one direction of a two-lane roadway). However, in viewing the adjacent lane from the primary lane, any distresses that are clearly visible should be marked on the distress forms to provide some overall indication of the presence of distress in those lanes. Because the adjacent lanes most generally will still be under traffic, no attempts should be made to enter those lanes.

Table I-4. Distresses to be evaluated during pavement distress surveys.

JCP Distresses	**CRCP Distresses**
Corner Breaks	Linear Cracking *(longitudinal and deteriorated transverse)*
Linear Cracking *(longitudinal and transverse)*	Map Cracking
Joint Seal Damage *(longitudinal and transverse)*	Scaling
Joint Spalling *(longitudinal and transverse)*	Polished Aggregate
Map Cracking	Popouts
Scaling	Blowups
Polished Aggregate	Transverse Construction Joint Deterioration
Popouts	Patch/Patch Deterioration
Blowups	Punchouts
Faulting of Transverse Joints	Spalling of Longitudinal Joints
Patch/Patch Deterioration	Water Bleeding and Pumping
Water Bleeding and Pumping	Longitudinal Joint Seal Damage

After the completion of the distress survey, the results can be tabulated in the office using the distress summary form (figure I-9).

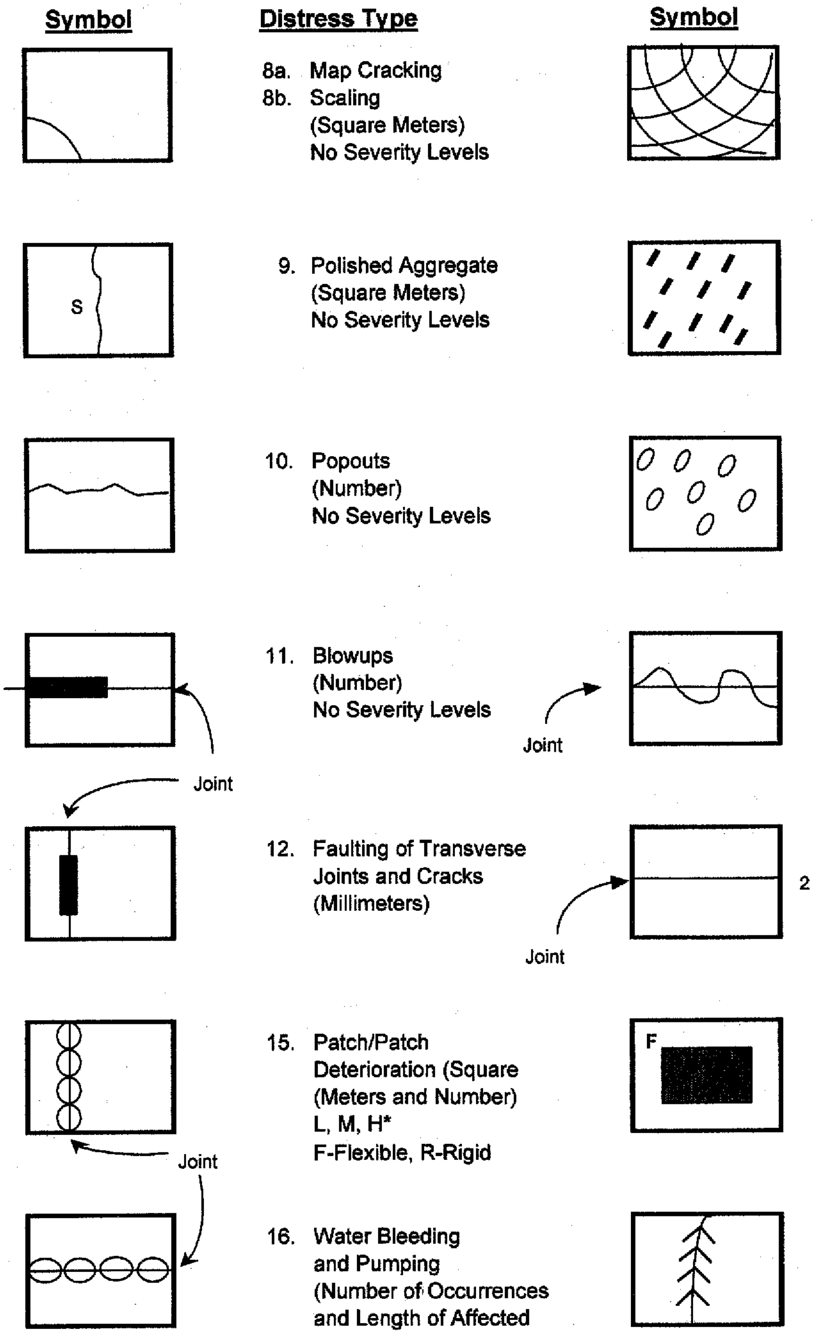
MRD Characterization

The goal of the field surveys is to provide information on the identification of the MRD type and causes. However, because of the similarity in the physical manifestations of many of the different MRD types, it is very difficult to positively identify the MRD during the conduct of the field survey. Indeed, it is believed that misidentification of various MRD types is a frequent occurrence because the diagnosis was based on the visual manifestations only. Thus, although strong indications of the type of MRD are possible from the field surveys, absolute identification is based on detailed laboratory analysis. It should be further noted that in some cases positive identification of the MRD type is not possible even after laboratory testing.

Although identification of the MRD will not be made during the field surveys, attempts will be made to characterize to the greatest extent possible the visible signs, or manifestations, of MRD. These manifestations include:

- Cracking pattern (location, orientation, extent, crack size).
- Staining (location, color).
- Exudate (presence, color, extent).
- Scaling (location, area, depth).
- Unusual surface texture (e.g., vibrator trails).

JCP DISTRESS MAP SYMBOLS



Distress Type

1. Corner Breaks
   (Number)
   L, M, H*

3. Longitudinal Cracking
   (Meters)
   L, M, H*
   S - Sealed

4. Transverse Cracking
   (No. of Cracks and Length)
   (Meters)
   L, M, H*

5a. Joint Seal Damage
   of Transverse Joints
   (Number)
   L, M, H*

5b. Joint Seal Damage
   of Longitudinal Joints
   (Number)
   No Severity Levels

6. Spalling of
   Longitudinal Joints
   (Meters)
   L, M, H*

7. Spalling of
   Transverse Joints
   (No. of Joints and Length)
   (Meters)
   L, M, H*

Distress Type

8a. Map Cracking
8b. Scaling
   (Square Meters)
   No Severity Levels

9. Polished Aggregate
   (Square Meters)
   No Severity Levels

10. Popouts
   (Number)
   No Severity Levels

11. Blowups
   (Number)
   No Severity Levels

12. Faulting of Transverse
   Joints and Cracks
   (Millimeters)

15. Patch/Patch
   Deterioration (Square
   (Meters and Number)
   L, M, H*
   F-Flexible, R-Rigid

16. Water Bleeding
   and Pumping
   (Number of Occurrences
   and Length of Affected
   Pavement)
   (Meters)
   No Severity Levels

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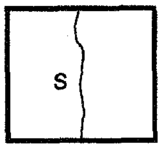
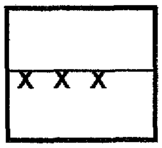
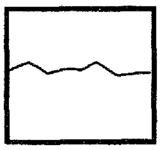
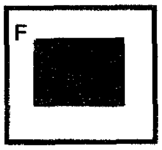
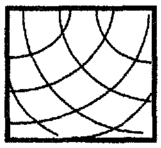
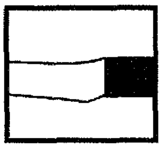
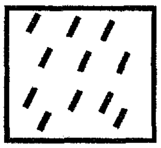
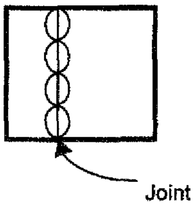
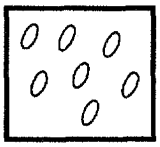
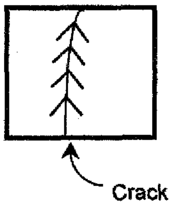
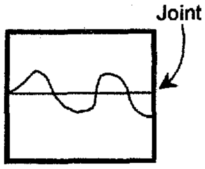
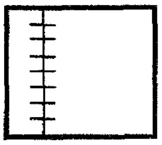
* Low, Moderate, and High Severity Levels

Figure I-12. Distress map symbols for jointed concrete pavements (adapted from SHRP 1993).

CRCP DISTRESS MAP SYMBOLS

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

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* Low, Moderate, and High Severity Levels

Figure I-13. Distress map symbols for CRCP (adapted from SHRP 1993).

By recording these factors during the field surveys, and then comparing them to the results of later laboratory analyses, it may be determined that certain visible signs are strong indicators of a specific MRD type, which will prove useful in future diagnoses. Figure I-11, presented previously, is used to record these critical factors.

The typical MRD cracking pattern will also be recorded during the field surveys using figure I-10. This is done for only a single typical slab to illustrate the prevailing distress pattern. Again it is noted that to facilitate the field survey, the MRD cracking is not drawn on the LTPP distress survey forms (only the general areas of distress are noted).

In observing the MRD crack patterns on the pavement, it may be helpful to spray the surface using a water bottle. This serves to make the fine cracking pattern much more visible as the pavement surface dries, while water still remains in the cracks.

Photo/Video Documentation

At the conclusion of the distress surveys, a complete photo or video summary of the inspection site is performed. The purpose of this photo summary is to document the condition of the pavement, as well as to record the prevailing foundation and drainage characteristics of the roadway. Either a photo log or video log may be conducted, although the best documentation may be captured using both technologies: the photo log for conveying pavement condition in reports and presentations, and the video log for providing a more complete record of the overall pavement condition.

The standard set of photographs shown in table I-5 should be taken of each project, but some variation is acceptable depending on the extent of distress. If the project is videotaped instead of photographed, the video should follow the same sequence of shots.

Table I-5. Standard photographs to be taken for each project.

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

DETERMINATION OF WHETHER FURTHER TESTING IS WARRANTED

Before proceeding with coring and an extensive laboratory evaluation, it is important to make a preliminary assessment of whether the observed pavement deterioration is the likely result of an MRD. The proposed assessment is based primarily on the results of the distress survey, in which pavement distress data was collected in accordance with the procedure described above. Figure I-14 presents the flow chart illustrating the assessment procedure. The various aspects of the flow chart are discussed below.

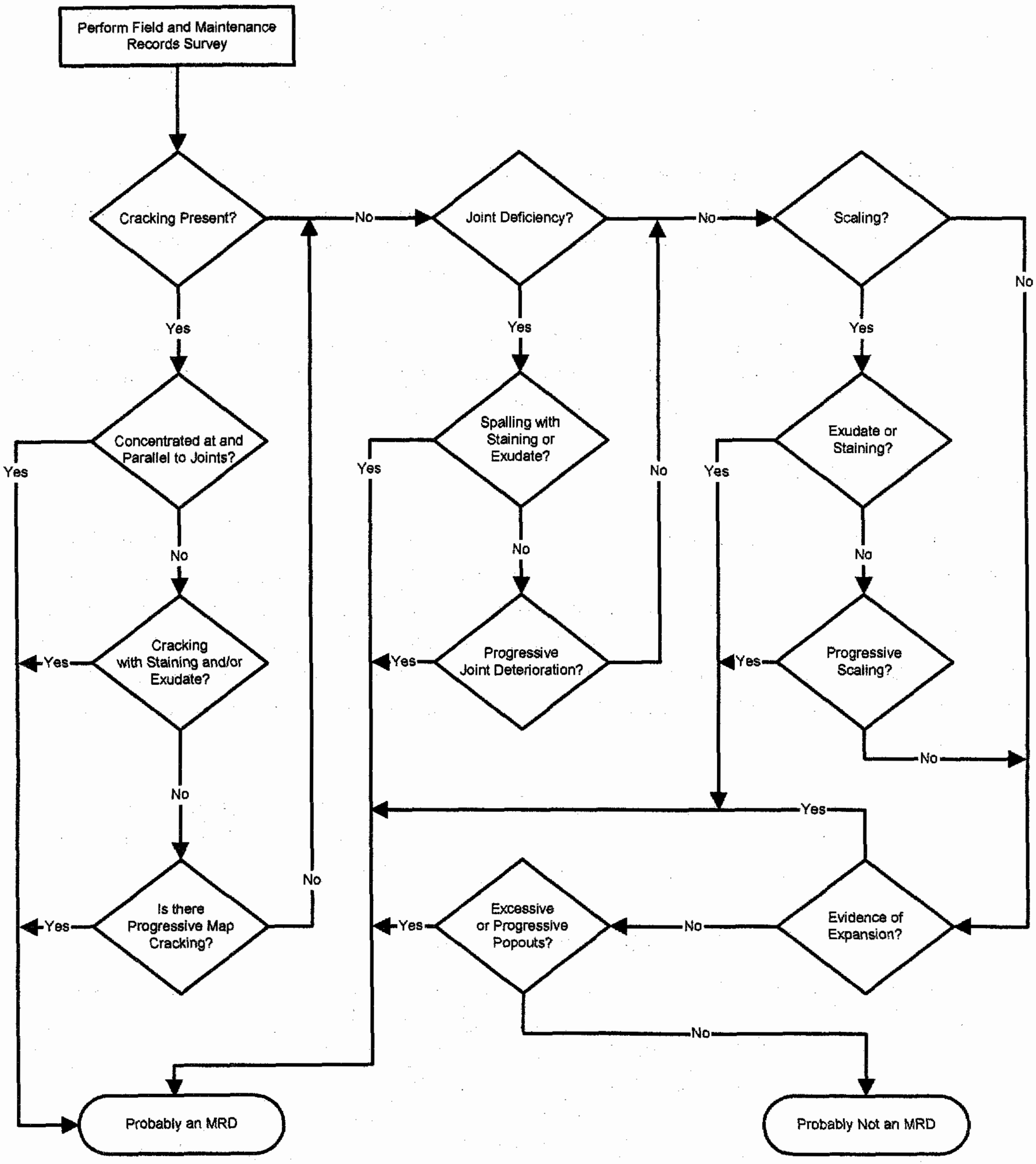


Figure I-14. Flow chart for assessing likelihood of MRD.

In accordance with the modified LTPP procedures described previously, distresses affecting concrete pavements can be classified into four general categories: cracking, joint deficiencies (jointed concrete pavements only), surface defects, and miscellaneous distress. Figure I-14 uses these distress categorizations to systematically progress through the assessment process. The first question posed is whether cracking is present. If it is, the next question is whether the cracking is concentrated at and parallel to the joints/cracks as drawn on figure I-10 and listed in figure I-11 (note this pattern is characteristic of D-cracking as described in the LTPP manual). If so, it is possible that an MRD is affecting the pavement and a coring/laboratory analysis should be conducted to determine the specific mechanism(s).

If the cracking is not concentrated at joints/cracks, but does have surface staining and/or exudate associated with it, it is possible that an MRD is at work. This information can be garnered from figure I-11. In this case, coring should be conducted to more fully assess the distress mechanism.

Of the joint deficiencies presented in the modified LTPP procedure, the one of greatest interest in assessing a pavement for MRD is spalling, particularly if staining and/or exudate is associated with it. Refer to figures I-9 and I-11 to determine if this is the case. If so, it is possible that an MRD is responsible for the observed distress and coring should be conducted.

The primary LTPP surface defect of interest is map cracking. If the map cracking is associated with staining and/or exudate, has become progressively more extensive and/or severe over time, and/or is associated with signs of pavement expansion, it is possible that an MRD is at work and a coring/laboratory evaluation is warranted. Scaling is another surface defect commonly associated with MRD. If an exudate is associated with observed scaling and/or the scaling has become progressively more pronounced over time, it is also possible that an MRD is responsible. Excessive popouts are also an indicator of an MRD.

In general, if it is concluded that an MRD is possible, a coring/laboratory investigation should be conducted. In some cases, the type, extent, and severity of distress might be such that engineering judgment should be exercised to determine whether the project should be more thoroughly analyzed at this juncture using coring/laboratory evaluation, or whether a “wait and see” approach is more appropriate. If the latter alternative is selected, the pavement should be visually monitored on a routine basis to assess whether distresses potentially associated with MRD are becoming progressively worse, at which point coring/laboratory evaluation should be conducted.

SAMPLING AND HANDLING PROCEDURES FOR MRD DETERMINATION

This section of the guideline presents recommended practices for obtaining and handling samples of existing PCC pavements for the purpose of diagnosing MRD. It describes a systematic approach to determine the numbers of samples required and appropriate locations from which to retrieve those samples. It also presents techniques for retrieving, handling and storing the specimens in a manner that both maintains the physical integrity and chemical composition of the material.

Sampling Existing PCC Pavement for Diagnosis of MRD

This section describes the procedures for conducting field sampling and testing of pavements that appear to be currently exhibiting MRD. Included in this discussion are preparation activities, sampling and handling procedures, shipping procedures, and quality control activities. The sampling procedures discussed are only for the positive identification of MRD, and are not designed for checking construction quality control. In all cases, the samples must be obtained in a manner that reflects the variation pattern in the concrete. In hardened concrete being analyzed for MRD, this could require taking random samples from within a panel to determine the variability of the development of MRD within the panel. Alternatively, sampling could be concentrated in areas of known or suspected MRD development in order to characterize the conditions that have led to that development. Thus, the recommended sampling plan presented in this section can be altered to reflect unique field conditions as long as a minimum number of specimens is obtained to allow for accurate characterization.

Preparation for Field Sampling and Testing

Organization, Supervision, and Coordination

The first essential activity in field sampling and testing is the establishment and continuation of communication between all involved parties with regard to such items as planning, scheduling, responsibilities, and safety. A planning meeting should be held to initiate this communication and coordination well in advance of any field operations. It should be attended by members of the local MRD research team [including laboratory test technicians and representatives of any disciplines directly related to the materials being tested (e.g., geologists, petrographers, cement and concrete engineers, soil scientists, and so on)]. Additionally, if in-house personnel are not performing sampling, representatives of the sampling contractor should also be present. Names, addresses, and phone numbers of appropriate personnel should be exchanged. The scope of work, general locations of the pavement test sections, schedules, duties and responsibilities of supervisory personnel, permits (if any), reports and field documentation, safety regulations and requirements, legal requirements, emergency actions, and other concerns should be discussed and resolved at this time (SHRP 1990).

Other information that should be provided to the sampling contractor or personnel includes:

- Maps showing specific locations of pavement test sections, materials stockpiles, or sources.
- Sampling locations and sample type(s).
- Expected conditions at each pavement test section, such as pavement type, layer thicknesses, and locations of utilities.
- Possible changes in the scope of work at any given field site, including the need for contractor-provided traffic control and test pit excavation.
- Standard forms for scheduling, data collection, and reporting of work accomplished.

Coordination requirements for traffic control, permits, insurance, and other legal requirements related to fieldwork vary between governing agencies. Field-testing and sampling contractors should note that adequate advance notice will be required by highway agencies for traffic control arrangements; traffic control and lane closures may have to be provided by the contractor in some cases (SHRP 1990).

Permits and Insurance

All pavement sampling work should be performed within the right-of-way owned by the governing highway agency. If drilling or sampling contractors are used, it should be their responsibility to familiarize themselves with all applicable regulations and secure all necessary permits to perform the work, including interstate shipping of samples and transportation of nuclear gauges, if appropriate. Drilling and sampling contractors, if used, should exercise caution to avoid underground or overhead utilities in the performance of their work, and they should be made responsible for all injury to persons, damage to property, either directly or indirectly, that may result from their operations. The governing highway agency should provide any drilling or sampling contractors with available information on underground utilities from their records (SHRP 1990).

Vehicles, Equipment, and Materials

The field sampling and testing contractor should, at all times, provide at the field site all vehicles, equipment, and materials required by the contract. On sites that require test pits, a pavement saw capable of cutting the full depth of inseparable bound pavement layers should be used. These items should be in proper working order and meet Federal, State and/or local regulations (as appropriate) with respect to configurations, load restrictions, and safety requirements. Drivers of the vehicles must meet State and local requirements with respect to the number of hours driven/worked within a 7-day period. Expendable items, such as bits, packaging materials, rope, etc., shall be available in quantities that will preclude any operational delays. Nonconformance should be considered cause for shut down of operations without compensation of any kind (SHRP 1990).

Traffic Control and Safety

Whenever drilling, sampling, and testing activities are being performed on highway pavements, the safety of the operating crews as well as the traveling public is of the utmost importance. A comprehensive safety plan must be developed and implemented throughout the drilling, sampling, and testing contract. The safety plan should conform to local and Manual on Uniform Traffic Control Devices (MUTCD) requirements with regard to reflectorized vests, hard hats, safety glasses, adequate clothing (including foot gear), and first aid equipment. Standard operating procedures for drilling, sampling, and testing should be prepared and field crews should be trained using these procedures.

All traffic control arrangements should be scheduled as far in advance as possible and should adhere to the guidelines provided in the MUTCD or the agency's governing requirements. Where traffic control activities are provided by the governing highway agency, any contractor field crews must be made aware of local safety requirements. When traffic control is provided by the contractor, procedures should be as required by the local governing highway agency. Work should be stopped and corrective actions taken when the crew chief or a designated highway agency official has any concerns about the safety of operations (SHRP 1990).

Visitors to the site should not be permitted within 8 m of drilling equipment. Vehicles in the work site should be facing in the direction of traffic to avoid any possibility of confusion. When a drill rig is facing the oncoming traffic, another vehicle should be located between the drill rig and the oncoming traffic and facing in the direction of traffic. Care should be exercised to minimize the encroachment of water from drilling operations onto the pavement surfaces open to traffic. In case of freezing conditions, deicing chemicals must be used to prevent the formation of ice on the traveled pavements and around the drilling operations (SHRP 1990).

Obtaining Hardened Concrete from *In situ* Pavements for MRD Investigations

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

ASTM C 823, *Standard Practice for Examination and Sampling of Hardened Concrete in Construction*, describes this, as well as many other aspects of sampling hardened concrete, in considerable detail. Other sources of information include Abdun-Nur (1970) and ACI 214, *Evaluation of Results of Tests Used to Determine the Strength of Concrete.* Standardization of this sampling process is difficult because it involves a great deal of judgment that is often difficult to standardize.

If the purpose of sampling hardened concrete is to assess overall properties and variability, it should be done randomly and objectively so that the samples taken are not weighted with unusually poor or unusually sound concrete. However, sampling in the course of investigations of MRD is different from quality control/assurance testing in that samples are commonly taken to exemplify unusual or extreme conditions or features that will aid in the identification of the causes of distress or failure of the concrete. It is this type of sampling that will be discussed below.

Sampling Plan (Number, Size, and Location of Samples)

The sampling plan must be developed with consideration of the results of field condition surveys that have been performed previously, which indicate whether or not the concrete is believed to be in similar condition and of similar quality. For the purposes of MRD testing, required sample numbers and sampling locations depend greatly upon the purpose of the study. In the vast majority of cases, evidence of distress will be obtained from the results of the visual assessment conducted in accordance with these guidelines. As such, targeted sampling is the appropriate approach for conducting an MRD evaluation used solely to characterize the type, severity, and extent of distress.

Selection of Coring Slabs: Data collected during the field survey will be useful in selecting coring locations. For example, information detailing the location and characteristics of the project should be reviewed by the person(s) developing the sampling plan as well as by the person(s) that will be executing the sampling plan prior to their visit to the field site. This information should include a detailed sketch or drawing of typical observed MRD (figure I-10), including the crack pattern and the presence of staining and any exudate. It will also include categorization of the MRD distress (figure I-11) in terms of its cracking pattern (i.e., location, extent, and size), staining (i.e., location and color), exudation (i.e., presence, color, and extent), scaling (i.e., location, area, and depth), and surface texture (i.e., visible trails, discoloration, distressed).

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

Coring slabs should always occur within the inspection site(s) evaluated during the visual assessment. In this way, the detailed crack maps produced during the visual survey can be used to select specific coring slabs. It is recommended that a minimum of one slab be cored within each site. A minimum of four cores will be obtained from each slab (depending upon the observed distress). Specific details concerning recommended core locations are presented in the next section.

Number of Samples Per Coring Slab: The number of concrete samples and the dimensions of the pieces retrieved in the sampling operation should conform with the stipulations of the tests to which the samples will be subjected. In some instances, two or more tests may be performed on a single sample, provided that the first test does not modify the properties of the concrete to be evaluated by subsequent tests, as described in Guideline II in this series of guidelines. One example of acceptable successive testing is the determination of air-void content by linear traverse or modified point count followed by petrographic examination. An unacceptable sequence would be to stain a specimen using uranyl acetate and then analyze it using the scanning electron microscope (SEM).

An adequate number of cores must be taken to represent the condition of the pavement, yet the number must be restricted due to practical restraints on resources, both for field coring and laboratory analysis. Figures I-15 and I-16 are the proposed sampling rate developed in consideration of the unique nature of targeted sampling used in an MRD investigation of JCP and CRCP, respectively. As can be seen, the minimum number of cores required per slab is somewhat dependent on the type(s) of distress observed.

Recommended core locations are presented in figures I-17 through I-20. If the distress in a JCP is concentrated at the joints, a minimum of four core samples is required if the pavement is doweled. This is illustrated in figure I-17, where Cores A and B are positioned directly over the joint, with the center of the 150-mm core barrel offset 50 mm from the middle of the joint. In this way, the core can be cut in the direction of traffic to analyze the concrete a distance of 125 mm from the joint interface. Core A is obtained directly over a dowel bar located between the wheel paths to assess the corrosion of embedded steel. Core B is obtained between dowel bars in an area affected by MRD. Core C is the third core obtained near the joint, located approximately 0.3 m from both the joint and outside lane stripe. This core will be used to assess the extent of deterioration away from the joint, and thus may or may not be in an area visibly affected by MRD. Core D is positioned in a center slab location free of observable distress to determine if distress is present in the microstructure even though it is not yet visible on the surface.

Figure I-18 shows the recommended coring locations for a JCP suffering from MRD that is not concentrated at the joints. Cores A through D are basically the same as previously described and an additional core, Core E, has been added. This core will be obtained over visible deterioration to assist in identifying its cause. This deterioration may be a crack (as is common in map cracking), spall, or delamination. In some cases, this core may be pulled from deteriorated vibrator trails. It is noted that Core D should be taken from a visually non-distressed area. In some severe cases of MRD, a non-distressed area might not be available, in which case the core can be taken from a deteriorated area or not at all.

Figures I-19 and I-20 present a similar pattern for CRCP. It is noted that instead of obtaining Core A through a dowel bar, it instead should be taken through the reinforcing steel. All other cores should be obtained between the steel, if possible, unless corrosion of embedded steel is the specific distress being investigated.

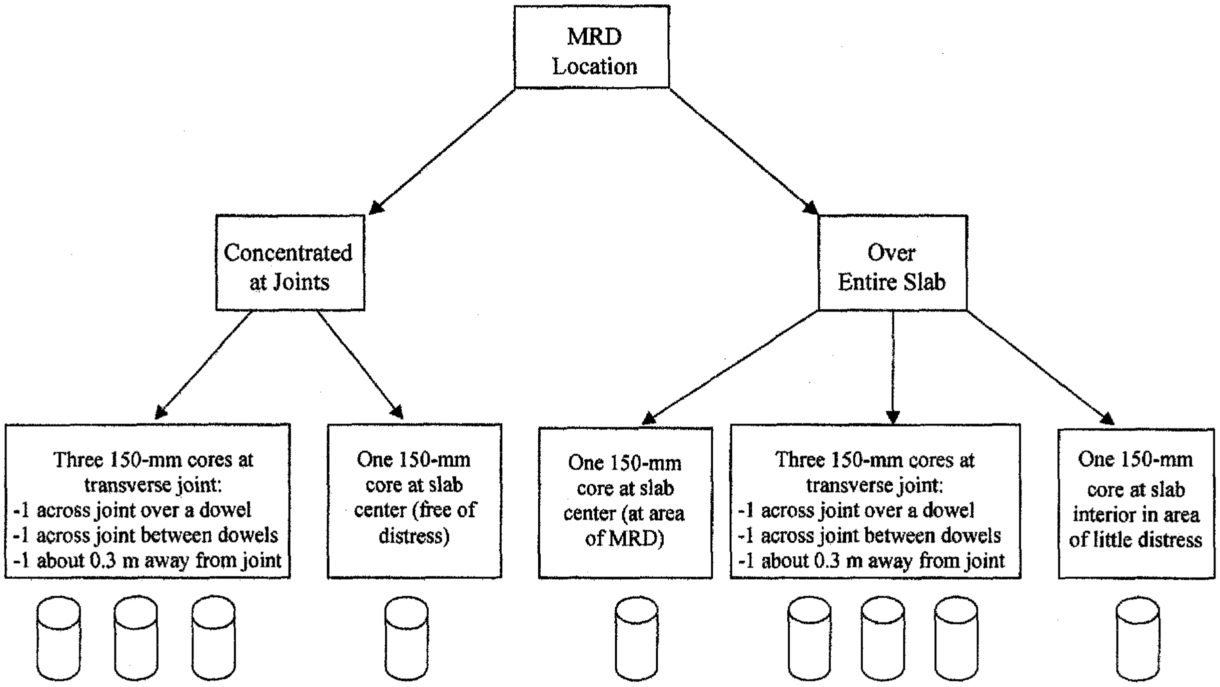


Figure I-15. Illustration of minimum number of cores for JCP required for MRD evaluation.

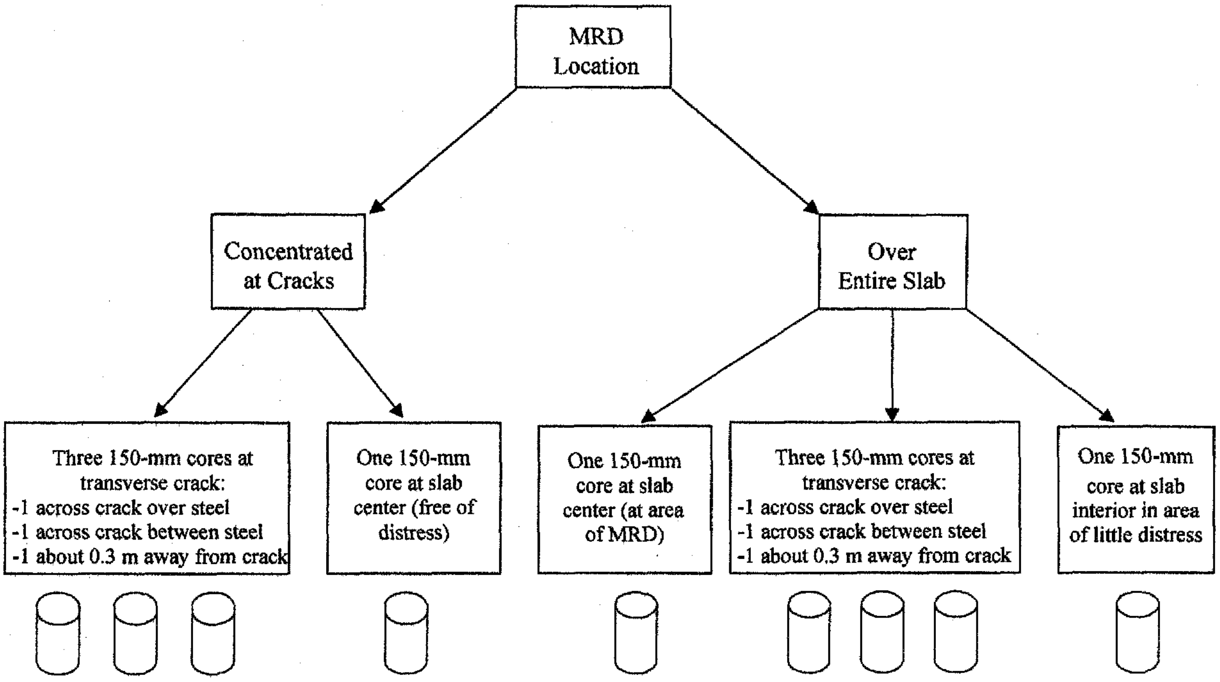
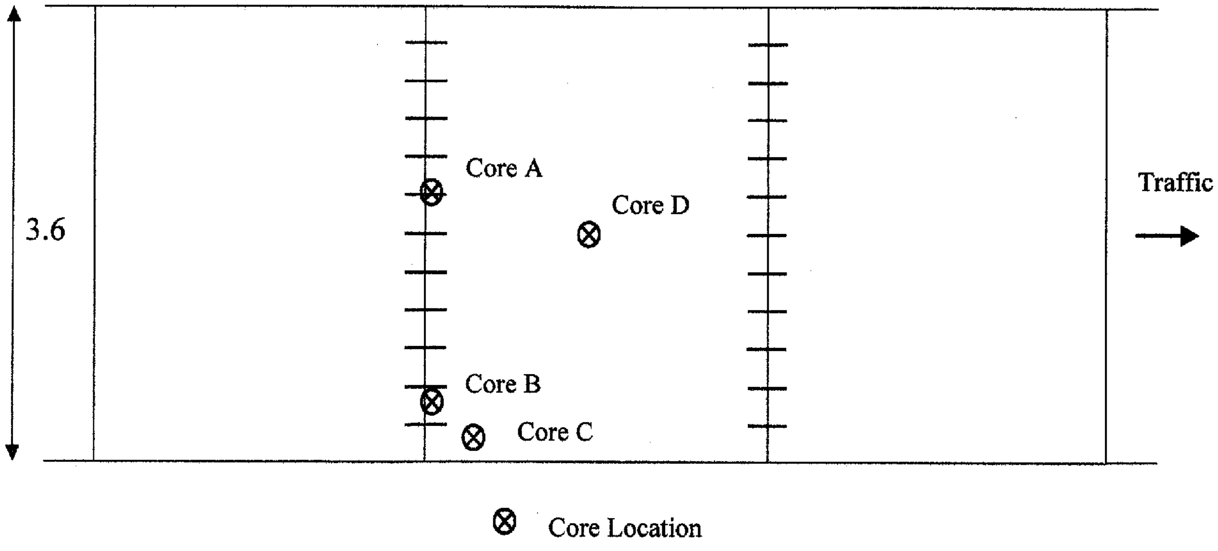


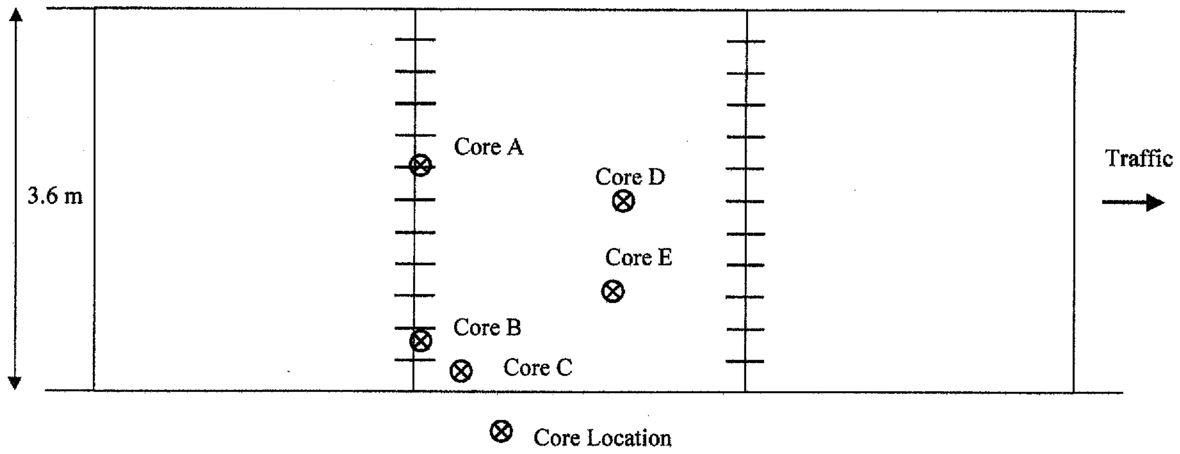
Figure I-16. Illustration of minimum number of cores for CRCP required for MRD evaluation.



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.

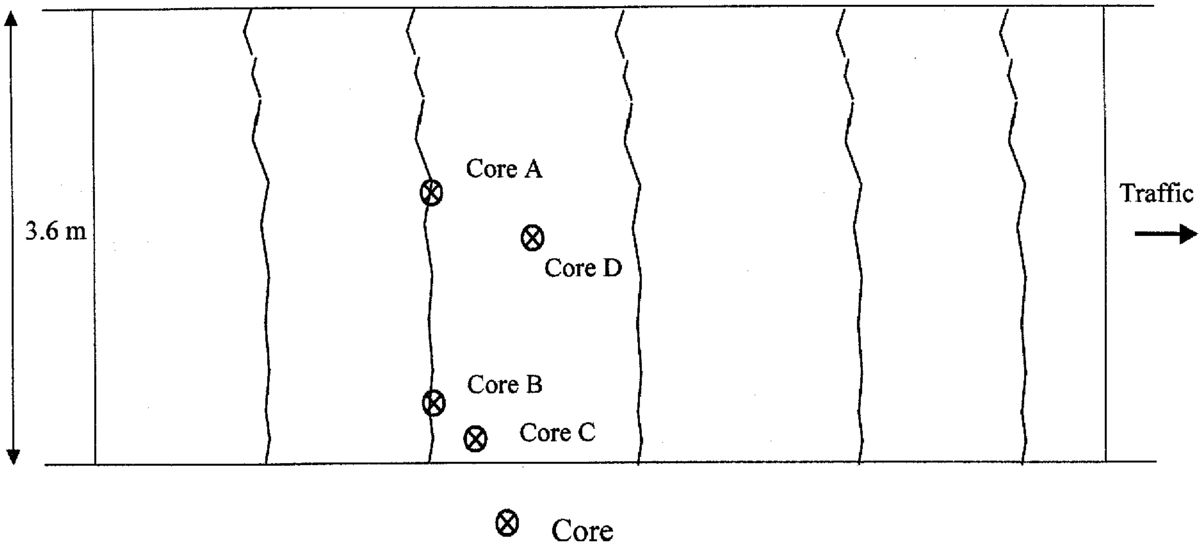
Figure I-17. Specific core locations for JCP with MRD concentrated at the joints.



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

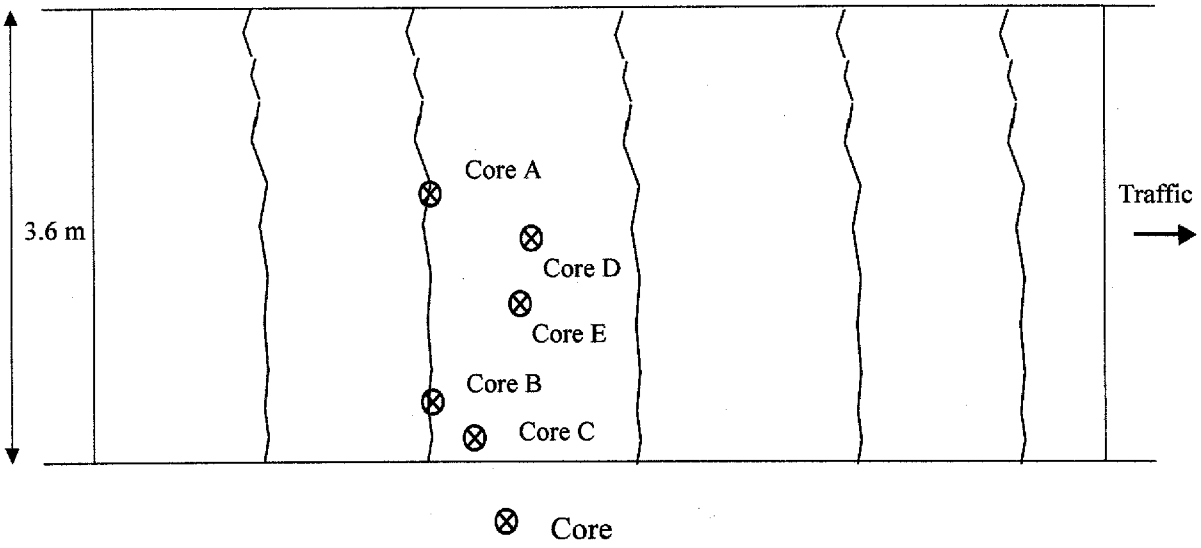
Figure I-18. Specific core locations for JCP with MRD not concentrated at joints.



Core Summary

Core A	Take across the crack and over steel near the middle of the lane, offset about 50 mm from center of crack.
Core B	Take across the crack and between steel, within 1 m of lane edge.
Core C	Take about 0.3 m away from crack and 0.3 from lane edge.
Core D	Take in the center of the lane in an area free of MRD.

Figure I-19. Specific core locations for CRCP with MRD concentrated at the cracks.



Core Summary

Core A	Take across the crack and over steel near the middle of the lane, offset about 50 mm from center of crack.
Core B	Take across the crack and between steel, within 1 m of lane edge.
Core C	Take about 0.3 m away from crack and 0.3 from lane edge.
Core D	Take in the center of the lane in an area of little MRD.
Core E	Take in the center of the lane in an area with MRD.

Figure I-20. Specific core locations for CRCP with MRD not concentrated at cracks.

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

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

Specimen Retrieval

Pavement surfaces should be marked at the coring location to indicate the direction of traffic before coring or sawing. The markings should be clearly visible and easily interpreted after the coring operation is complete.

Procedures that cause mechanical or thermal damage to cores should not be used in the retrieval of cores or beams intended for shipment to the laboratory.

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

Information concerning the location, manner of retrieval, and field condition of all hardened concrete samples should be recorded for future reference. A typical hardened concrete sampling log sheet that may be useful for a coring site is presented in figure I-21.

Labels that clearly identify the core (including the coring site, slab number, core location, and date of coring) should be prepared and affixed to each sample. An example identification label is shown in figure I-22.

Special Considerations in Sampling Hardened PCC

Air-Void Samples: Whenever it is suspected that the entrained air system of hardened PCC is not sufficient to provide protection from freezing and thawing deterioration, or whenever it is suspected that the cause of low strength might be excess air content, air-void system measurements may be required. If the air-void system of the entire concrete placement is suspect, sampling should be performed in accordance with the instructions detailed in ASTM C 457:

LABORATORY LOG OF PCC PAVEMENT CORES

Sheet ____ of ____

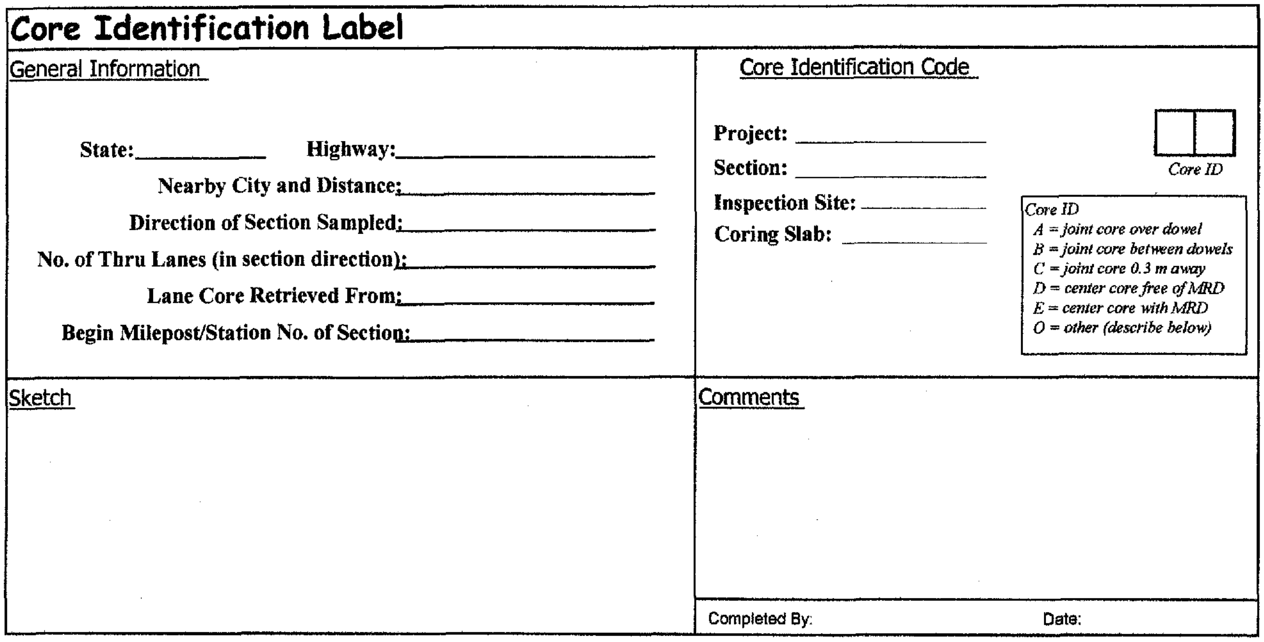
Project Designation: _______________ State: ______________________ Highway: ______________
Nearby City and Distance: ______________________________________ Direction: ______________
No. of Through Lanes (in direction sampled): ______________________ Lane Sampled: __________
Beginning Milepost/Station: _____________________ End Milepost/Station: ____________________

Operator: ___________________________ Core Diameter: _____________________________
Coring Date: _________________________ Core Barrel Tip Type: ________________________
Job ID: _____________________________

Note: Each column shown below should be used to record information for all cores/pieces extracted from a single panel. "Depth" should be measured from the pavement surface to the bottom of the core/piece and recorded to the nearest 2 mm. Front direction is the direction of traffic.

CORE NUMBER	A	B	C	D	E	OTHER
CORE ID						
LOCATION/STATION						
Offset From Joint						
Offset From Shoulder						
Number of Pieces?						
Core Piece No. 1						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 2						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 3						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 4						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
REMARKS						

Figure I-21. Typical hardened concrete sampling log sheet.



Core Identification Label

General Information

State: ________ **Highway:** ________
Nearby City and Distance: ________
Direction of Section Sampled: ________
No. of Thru Lanes (in section direction): ________
Lane Core Retrieved From: ________
Begin Milepost/Station No. of Section: ________

Core Identification Code

Project: ________
Section: ________
Inspection Site: ________
Coring Slab: ________

☐☐ *Core ID*

Core ID
A = joint core over dowel
B = joint core between dowels
C = joint core 0.3 m away
D = center core free of MRD
E = center core with MRD
O = other (describe below)

Sketch

Comments

Completed By:

Date:

Figure I-22. Example core identification label.

> *To determine the compliance of hardened concrete with requirements of specifications on the air-void content or specific surface and spacing factor of the void system, a sample of the concrete should be obtained from at least three locations in the body of concrete and microscopical measurements should be made on at least one section prepared from each of at least three of such samples.*

The three locations within the entire body of the placement under study from which samples must be determined in accordance with a rigorously developed random sampling plan, without regard for areas of extreme deterioration. Areas exhibiting specific features of interest should be sampled separately. Each sample should be large enough to allow the petrographer to prepare at least the minimum area of finished surface given in ASTM C 457, table 1.

Frozen Concrete: If it is suspected that the subject concrete was frozen before gaining adequate strength to resist freezing damage, at least one specimen should be obtained from the edge of the placement, from up against the form, or from a place exposed to the ambient temperature. It is in such an exposed area that the casts of ice crystals will form first. If companion cylinders were cast and cured similarly to the placement, they may show ice crystal casts on the surface in contact with the mold. Exposure of fresh concrete to ambient freezing temperatures usually affects the wearing surface only if the curing material is insufficient to retain the heat generated by the hydration of the cement or if the curing material is prematurely removed (possibly by wind).

Handling and Shipping of PCC Field Samples

Established sampling plans, methods, and procedures are often not enough to ensure adequately intact samples reaching the laboratory for the preparation of specimens for testing. Proper handling and care of samples in the field and in storage and shipping are essential if test results obtained from specimens prepared from these samples are to be meaningful and if reliable conclusions are to be drawn from such results.

Generally accepted field handling procedures include:

- All core or beam surfaces should be rinsed with fresh water to remove coring or cutting slurry prior to wrapping or packing for transport and/or shipping.
- If layers of a core sample separate during field handling, they should be rematched and secured with tape in the field prior to final wrapping.
- If the coring operation yields one or more core fragments, these should be shipped to the lab for visual and possible petrographic examination along with the core or larger core fragments.
- Identification labels (e.g., figure I-21) should be prepared and affixed that clearly identify the core, including the project number, section, inspection site, coring slab, core location, and coring date.

There are generally no limits on the duration of storage for hardened concrete specimens, provided that the desired storage conditions (if any) are maintained with respect to specimen moisture and temperature, exposure to atmospheric conditions, and so on. These conditions should be established such that the condition and composition of the specimen represent, as closely as possible, the condition of the field concrete at the moment of retrieval.

Shipping of Samples

In general, samples must be adequately identified and shipped in clean, strong containers. If the moisture content of the sample is important, the container must be moisture-tight. Containers should always be clean and free of any potential contaminants.

Shipping containers should be labeled with suitable rot- and mildew-resistant identification tags containing all of the information deemed pertinent so that field reporting, laboratory logging, and test reporting may be facilitated. Duplicate labels should be placed inside of the containers. If the containers have removable tops, the labels should be attached to the body of the container to avoid possible mislabeling of a sample if the container lids are accidentally switched.

Care should be taken to prevent damage to concrete specimens during transport and shipping. Intact cores or large core fragments can generally be transported from the field and shipped over short distances without damage when wrapped in plastic bubble wrap packing material and placed in appropriately sized plastic cylinder molds with lids. Similarly, sawed beams can also be wrapped in plastic bubble wrap packing material and shipping tape and placed in protective tubs or containers for transport from the field and shipping over short distances.

Specimens that will be shipped by common carrier should be afforded additional protection from damage. While there is no absolute way to prevent damage during shipping by common carrier, the following techniques and considerations may be helpful:

- Limit the size and weight of the loaded shipping container. For example, 20-L plastic pails with handles and sealable lids provide relatively inexpensive, easily handled shipping containers that generally cannot be loaded to contain more than 35 kg of construction materials.
- Large plastic storage tubs or milk crates have also been used to successfully ship concrete beams and cylinders. In single-use applications, beams and cylinders can be placed on a soft packing layer and foamed into place using commercially available spray packing foams. For repeated uses (e.g., frequent exchange of specimens between labs), tubs or crates have been fitted with foam layers containing cut-out areas that match the size of specimens that will be commonly shipped.
- Thicker layers of packing material should be used between the specimens and the bottom and sides of the containers. Most potential impact damage during shipping will occur in these areas.
- Any loaded containers weighing more than 35 kg should be clearly labeled as "HEAVY."
- All shipping containers should also be marked for proper orientation during transport (i.e., "THIS SIDE/END UP") and for proper care as well (i.e., "FRAGILE").
- If the specimens will be shipped to, from, or through an area experiencing subfreezing temperatures and protection from freezing is desired, provide additional insulation/packing and consider using expedited overnight carrier service. Trucking or other ground-based service may result in several days' exposure to varying climatic conditions.
- If the specimens will be shipped from an area under agricultural quarantine, the United States Department of Agriculture should be consulted to ensure that contaminated soil is not transported.
- If it is desired to maintain specimens in a moist condition, they should be wrapped in wet cloths or towels, then wrapped in plastic "cling wrap" and finally taped using shipping tape to provide a moist, sealed environment before wrapping in plastic bubble packing for protection.

SUMMARY OF FIELD PROCEDURES

The distress survey, which provides a systematic procedure for conducting distress surveys of pavements exhibiting MRD, not only establish the overall condition of the pavement, but also a means to characterize the manifestations of MRD occurring on the pavement surface. Table I-2 summarizes the important parts of the distress surveys. After the conduct of the distress survey, the next step is to determine whether coring is warranted and, if it is, to identify testing locations and the actual material sampling for later testing in the laboratory.

This guideline then presents procedures for obtaining and handling hardened concrete samples for use in identifying the presence of MRD in PCC pavements. It describes a systematic approach to determining the numbers of samples required and appropriate locations from which to retrieve those samples, as well as techniques for retrieving, handling, and storing the specimens in a manner that maintains the physical integrity and chemical composition of the material. Compliance with the proposed guideline will help to ensure that subsequent laboratory tests and analyses provide accurate and meaningful indications of MRD presence or potential.

GUIDELINE II — LABORATORY TESTING, DATA ANALYSIS, AND INTERPRETATION PROCEDURES FOR DISTRESSED CONCRETE PAVEMENTS

This chapter presents the second of the three guidelines prepared as part of FHWA Project No. DTFH61-96-C-00073 entitled *Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements.* This guideline is divided into two major sections. The first section addresses the laboratory procedures used to examine concrete specimens and presents a systematic method of data collection, recording, and reporting. The second section of this guideline presents a systematic approach for interpreting the laboratory data, along with design, construction, field evaluation, and performance data, to make a diagnosis as to what distress mechanism(s) may be present.

LABORATORY TESTING

Overview

A key step in identifying the cause of a materials-related distress (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.

This section of the guideline is designed to be an informational guide for technical personnel and presents laboratory methods available for identifying MRD in concrete pavements using petrography, SEM, and other techniques. It also provides a brief description of commonly applied procedures and a suggested format for gathering and storing data. This manual is intended for both engineers and analysts where an engineer is defined as the person presenting the problem to the laboratory and the analyst is the laboratory personnel charged with diagnosing the concrete problem. In many cases, this may be the same person but often, they are members of a team of people that need to communicate clearly to arrive at a cause, or causes, for the suspected MRD. In this sense, both engineers and analysts must have a clear understanding of the required tests and the possible caveats associated with each test to properly interpret the results.

This section is organized starting with a brief description of recommended literature that will provide background information. This is followed by an overview of the approach to laboratory testing and a recommended data collection protocol using standard data collection forms. Last, an approach for data analysis and interpretation is presented. More detailed descriptions of the equipment and methods used to conduct optical microscopy, scanning electron microscopy (SEM), x-ray diffraction (XRD), and chem-laboratory analysis are presented in appendix B.

Recommended Literature

A number of valuable reference materials exist that provide an excellent presentation of the current body of knowledge related to methods of performing or interpreting diagnostic tests on concrete. Below are brief descriptions of a small selection of publications that provide an excellent review of various aspects of concrete petrography. Each publication covers a different aspect of forensic concrete analysis applicable to MRD identification and it is recommended that anyone charged with the analysis of concrete become familiar with these publications. This is not intended to be a complete list of all available publications discussing concrete analysis or diagnosis of deterioration. Instead, it is a list of publications that provide the average analyst with the required information to become proficient at analyzing concrete.

Rock and Mineral Identification for Engineers (FHWA 1991) is an excellent resource for rock and mineral identification. The purpose of this handbook is to summarize the distinctions between different types of rocks commonly used as aggregates and aid in their identification by engineers in the field or laboratory. Easily performed simple tests not requiring special facilities are introduced. The key information provided in the manual is summarized in a laminated pocket sized flowchart that is easily carried in the field. This publication is an excellent summary of fundamental petrology and can serve as a good primer for non-geologists who need to identify different rock types. Also, the role of aggregates in various types of MRD is briefly discussed.

An excellent manual on petrographic methods is entitled, *Petrographic Methods of Examining Hardened Concrete: A Petrographic Manual* (Walker 1992). This manual provides a detailed summary of the fundamental methods of concrete analysis including chapters on equipment and materials, sample documentation, preparation of specimens, and the observation of cracks, voids, and paste using the stereo and petrographic optical microscopes. In addition, other concrete constituents and parameters are discussed including mineral admixtures and water-to-cement (*w/c*) determination. Alkali–aggregate reactivity (AAR) is discussed and numerous photographs and micrographs are included to illustrate the different types of distresses. Various types of optical microscope methods are presented including a section on fluorescence microscopy techniques. Numerous color pictures of various concrete specimens are included and overall, detailed steps on the fundamental tests for concrete analysis are presented. It is recommended that anyone new to the concrete sample preparation and microscopy methods obtain a copy of this manual for reference purposes.

Another useful concrete petrography publication is *Concrete Microscopy* (Roy et al. 1993a). It provides a detailed description of the fluorescence microscopy method, including numerous micrographs showing various features as seen in fluorescence microscopy and the effects of poor sample preparation. Detailed instructions on the method of gathering and the interpretation of data are included.

A third valuable publication on the subject of concrete petrography is *Concrete Petrography* by St John et al. (1998). This excellent textbook discusses concrete petrographic techniques, providing information and references along with details of specific techniques. The text also discusses defects in, and deterioration of, concrete and provides a wide range of micrographs illustrating common microstructural features. Also, details of optical and chemical properties of phases found in concrete are presented.

Significance of Tests and Properties of Concrete and Concrete-Making Materials [American Society for Testing and Materials (ASTM) 1994] covers a broad range of concrete topics, addressing virtually all tests of concrete and mechanisms of concrete distress and deterioration. The compilation is laid out in six general sections; General Information, Freshly Mixed Concrete, Hardened Concrete, Concrete Aggregates, Other Concrete Making Materials, and Specialized Concretes. A well-known expert provides a very detailed and complete discussion of the issues under each section. Each chapter has extensive references allowing for more detailed examination of any aspect presented. It is an excellent reference book and a recommended addition to the laboratory library.

There are a large number of ASTM standards that address laboratory practices relative to concrete. In general, three of these standards directly address the examination of hardened concrete and should serve as a basis for any laboratory examination of concrete exhibiting MRD. The applicable standards are ASTM C 457 *Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete*, ASTM C 823 *Practice for Examination and Sampling of Hardened Concrete in Constructions*, and ASTM C 856 *Practice for Petrographic Examination of Hardened Concrete*. One important impact of these standards is in the case of litigation where the credibility of laboratory results will be based in part on adherence to these accepted standards. Unfortunately, as detailed procedures, these ASTM standards often fall short, forcing the engineer to make decisions about methods without sufficient background information.

Approach to Laboratory Analysis of Concrete

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), SEM, analytical chemistry, and XRD.

Optical microscopy using the stereo microscope and the petrographic microscope are the most versatile and widely applied tools for diagnosing causes of MRD. Electron microscopy is becoming more prevalent, especially for chemical identification of reaction products and other secondary phases using energy dispersive spectroscopy (EDS). Analytical chemistry plays a very important role in determining key parameters of the concrete (e.g., *w/c*, chloride content). XRD is applied in some cases but is not widely used in the analysis of 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/c*. Other techniques, such as SEM and XRD, have also been widely used by researchers and are becoming more common for forensic investigations of concrete failures.

When initiating the study of deteriorated concrete, or any material, an analysis plan of how to approach the problem must be followed. This plan very often reflects a process of elimination; rather than proving what the 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 II-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 core visually, a hand lens or stereo microscope can be used to look more closely at pertinent features. In some cases, as a result of this visual and stereo microscope analysis, the probable or certain cause of distress is identified. In most cases, a few potential MRD types can be eliminated and further analysis can then focus on those remaining. At this point, the analyst must decide which examination technique can be performed to confirm a given MRD, or eliminate other MRD types, thereby narrowing the choices to the most probable mechanism.

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 quite common to view the same specimen in all three instruments. Staining in particular can assist in the optical evaluation although it may interfere with SEM analysis.

As of this writing, a SEM is not available in most State highway agency (SHA) laboratories. However, laboratories trying to actively diagnose and treat MRD in concrete should consider either the purchase of an SEM or establishing cooperative arrangements with other institutions to gain access to an SEM. Although a trained petrographer can use the petrographic microscope and identify practically all minerals and aggregate reaction products present in a concrete specimen, this often requires thin section preparation or detailed analysis of picked grains with refractive index liquids. This requires a highly skilled concrete petrographer to analyze and interpret the complex and vast array of information revealed by the petrographic OM. In contrast, the SEM is an instrument 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.

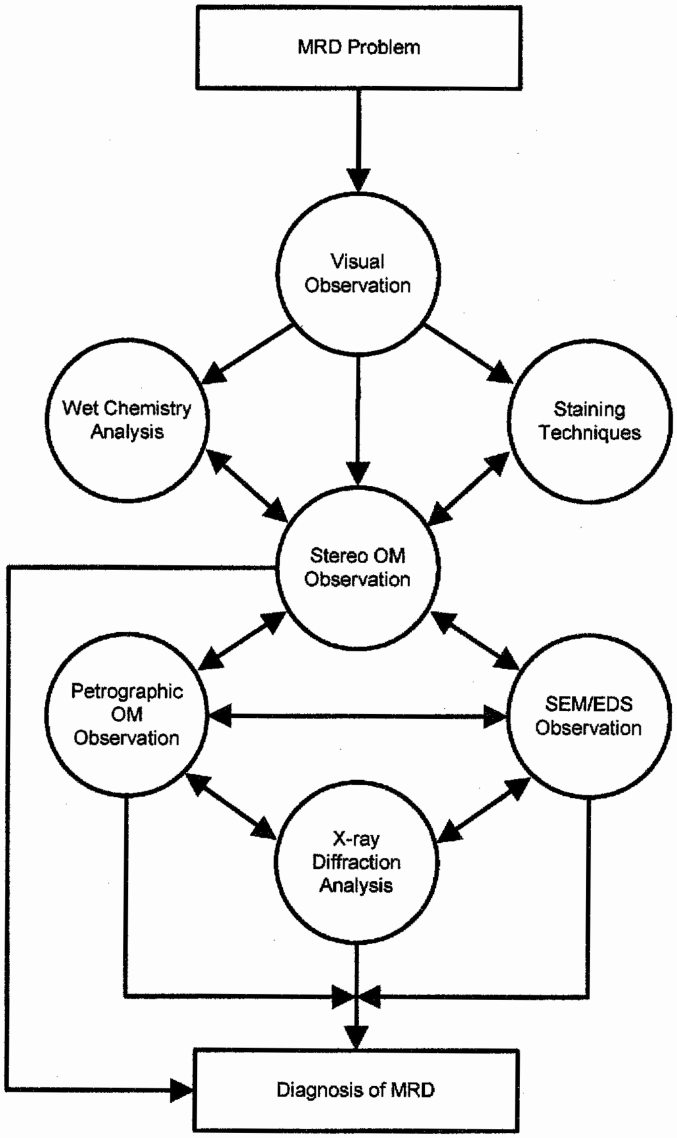


Figure II-1. Fundamental process for analyzing a concrete MRD sample.

Both x-ray mapping and x-ray microanalysis are very useful ways of identifying components of the microstructure. However, the use of the SEM has some disadvantages. 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 II-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.

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

Procedures for the Analysis of Concrete

The purpose of this section is to provide an overview of the analytical procedure used to examine MRD in concrete. To accomplish this standard analysis, data sheet templates designed to follow a sample (most commonly a core) through the laboratory evaluation are provided. The use of these data sheets will facilitate laboratory analyses and data interpretation in a number of ways. First, these data sheets provide a framework for the "questions you ask about the material" and are intended to be consistent with the requirements of ASTM C- 856 *Practice for Petrographic Examination of Hardened Concrete*. It is noted that the individual questions and data queried in these forms do not constitute the full implementation of ASTM C- 856. Instead they focus on the analysis of deteriorated pavement concrete, which is only a small fraction of what this very broadly applied Practice addresses.

Another benefit of using data forms is that they serve as the "lab notebook" for recording and archiving laboratory results. As an example, careful inspection with a stereo microscope alone can often reveal the reason for concrete failure. Although a diagnosis of MRD based on stereo microscope observations may be correct, it is often necessary to use other techniques to get confirmation. The data sheets provide a systematic way of gathering and archiving the results of multiple laboratory analyses that may be conducted by multiple technicians, often in different laboratories. Finally, personnel outside the laboratory want to know that a core was examined for all possibilities and no rush to judgment was made on limited data. By using standard data collection forms, the diagnostic indicators laboratory personnel were looking for will be clear to others. When tests are not performed, it should be noted on the data sheet along with an explanation of why the test was not conducted. This reporting protocol will help provide an understanding of the decisions made by laboratory personnel to arrive at a diagnosis of material failure. The data sheets provided are separated into the following major categories roughly corresponding to the steps illustrated in figure II-1:

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

Within each data sheet, tests and diagnostic features are presented in a tabular format along with possible or common results. It is up to the analyst to observe and make judgments about the concrete relative to the questions posed on each data sheet. Laboratory personnel should complete only those data sheets that apply to analyses conducted. These observations are combined with all other data to arrive at a diagnosis of the MRD observed.

Analysts are expected to enter data and observations on these forms and a copy of these forms accompanies each core, following it through the analysis process. Additionally, completed forms should be provided to the engineer along with the final laboratory report. Copies of these forms are provided in appendix C. The groupings and individual forms are as follows:

Group 1: Core Receipt and Cataloging
- Figure II-2. Laboratory Log of PCC Pavement Cores.
- Figure II-3: Core Sampled for Laboratory Analysis.

Group 2: Visual Inspection
- Figure II-4: General Condition of Concrete.

Group 3: Stereo Optical Microscope Examination.
- Figure II-5: Observations of the Concrete.
- Figure II-6: Alterations of the Aggregates.
- Figure II-7: Results of ASTM C457.

Group 4: Staining Tests
- Figure II-8: Summary of Staining Tests.

Group 5: Petrographic Optical Microscope Examination
- Figure II-9: Observations of the Concrete.
- Figure II-10: Alterations of the Aggregates.

Group 6: Scanning Electron Microscope Examination
- Figure II-11: General Conditions.
- Figure II-12: Summary of Scanning Electron Microscope Analysis.

Group 7: Chemical Tests
- Figure II-13: Summary of Chemical Tests.

Group 8: X-ray Diffraction Analysis
- Figure II-14: General Conditions.

LABORATORY LOG OF PCC PAVEMENT CORES

Sheet ____ of ____

Project Designation: ______________ State: ____________________ Highway: ____________
Nearby City and Distance: ______________________________________ Direction: ____________
No. of Through Lanes (in direction sampled): ____________________ Lane Sampled: ________
Beginning Milepost/Station: ____________________ End Milepost/Station: ______________

Operator: ___________________________ Core Diameter: ___________________________
Coring Date: _________________________ Core Barrel Tip Type: ______________________
Job ID: ______________________________

Note: Each column shown below should be used to record information for all cores/pieces extracted from a single panel. "Depth" should be measured from the pavement surface to the bottom of the core/piece and recorded to the nearest 2 mm. Front direction is the direction of traffic.

CORE NUMBER	A	B	C	D	E	OTHER
CORE ID						
LOCATION/STATION						
Offset From Joint						
Offset From Shoulder						
Number of Pieces?						
Core Piece No. 1						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 2						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 3						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 4						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
REMARKS						

Figure II-2. Group 1: Laboratory log of PCC pavement cores.

Analyst	Core taken by	Date Cored
Date	Core ID	Job ID

CORE SAMPLED FOR LABORATORY ANALYSIS

Location (circle one): A B C D E Other?:	Picture: Top View
Core Diameter: mm	
No. of Pieces:	
Ht. Piece #1: mm	
Wt. Piece #1: kg	
Ht. Piece #2: mm	
Wt. Piece #2: kg	
Ht. Piece #3: mm	
Wt. Piece #3: kg	
Ht. Piece #4: mm	
Wt. Piece #4: kg	
Total Ht.: mm	
Total Wt.: kg	
Sketch	Picture: Side View

Figure II-3. Group 1: Core sampled for laboratory analysis.

Analyst	Job ID
Date	Core ID

Visual Inspection - General Condition of Concrete

Attach additional documentation and photographs as needed.

Diagnostic Feature	*Options*		*Comments*
Ring when struck lightly with a hammer?	Yes	No	
Does it break with your fingers?	Yes	No	
Is the concrete well consolidated?	Yes	No	
Is segregation apparent?	Yes	No	
Orientation/parallelism of aggregates?	Yes	No	
Visible surface deposits or exudate?	Yes	No	
Are cracks apparent in the paste?	Yes	No	
Are the cracks widespread?	Yes	No	
Cracks through aggregates?	Yes	No	
Are there cracks around aggregates?	Yes	No	
Deposits in cracks?	Yes	No	
Embedded items present?	Yes	No	
Adequate cover over the embedded items?	Yes	No	
Are embedded items corroded?	Yes	No	
Underside voids on elongated aggregates?	Yes	No	
Are air voids filled?	Yes	No	
Does paste hardness seem normal?	Yes	No	
Is paste hardness uniform throughout?	Yes	No	
High paste content?	Yes	No	
Gradation of aggregates?	gap	uniform	
Coarse aggregate top size?			
Coarse aggregate type?	crushed	natural	
Coarse aggregate rock type?			
Fine aggregate type?	crushed	natural	
Fine aggregate rock type?			
Alteration/reaction with aggregates?	Yes	No	

Comments

Figure II-4. Group 2: General condition of the concrete.

Analyst	Job ID
Date	Sample ID

Stereo OM - Observations of the Concrete

Attach additional documentation and micrographs as needed.

Diagnostic Feature	**Options**	*Comments*
Bleeding?	bleed channels pockets under aggregates	
Fractures?	none through aggregates in paste around aggregates	
Air void infilling?	yes partial no	
Air void shape	spherical irregular ellipsoidal	
Embedded item condition?	good corroded	
Surface condition?	cracked carbonated weathered leached normal	
Aggregate reaction products?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste	
Gaps around coarse or fine aggregates?	description estimate occurrence	
Gap widths larger for larger particles?	yes no	
Are the gaps filled?	yes no	
Coarse aggregate type?	gravel quarried crushed gravel slag other	
Lithological types of coarse aggregate?	specify types identified	
Orientation/parallelism of coarse aggregate?	specify which and specify direction of orientation	
Fine aggregate type?	natural other *(specify)* crushed	
Lithological types of fine aggregate?	specify types identified	
Paste color – *note uniformity of color.*	white light gray dark gray yellow green other *(specify)*	
Paste hardness – *note uniformity of hardness.*	soft hard medium	

Comments ____

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Figure II-5. Group 3: Stereo OM - observations of the concrete.

Analyst	Job ID
Date	Sample ID

Stereo OM Observations - Alterations of the Aggregates

Attach additional documentation and micrographs as needed.

Coarse Aggregates

Diagnostic Feature	*Options*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution or softening of aggregates?	yes no	
Reaction rims?	yes no	
Reaction products from alteration?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	

Fine Aggregates

Diagnostic Feature	*Options*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution or softening of aggregates?	yes no	
Reaction rims?	yes no	
Reaction products from alteration?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	

Narrative Description of Alteration ______________________________________

____

Figure II-6. Group 3: Stereo OM observations - alterations of the aggregate.

Analyst	Job ID
Date	Sample ID

Results of ASTM C 457

Method Used:	Length Traversed
	Area Traversed
Magnification	No. of Stops

Air-Void System Parameters			
Specification	Typical Range for Acceptable Air Entrained Concrete	Calculated Value for Original Air-Void System *(infilling counted as air)*	Calculated Value for Filled Air-Void System *(infilling counted as paste)*
Spacing Factor ($\bar{L}$)	0.01 - 0.02 mm		
Specific Surface (a)	23.6 - 43.3 mm^{-1}		
Paste/Air Ratio	4 - 10		
Void Frequency (n)			

Phase Abundance Analysis	
Phase	**Volume Percent**
Coarse Aggregate	
Fine Aggregate	
Paste	
Original Air Content	
Filled Voids	

Comments____

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Figure II-7. Group 3: Results of ASTM C 457.

Analyst	Job ID
Date	

Sulfate/ASR Reaction Product Staining *Attach additional documentation and micrographs as needed.*			
Core ID	*Method Used*	*Positive Staining*	*Comments*
		Yes No	
		Yes No	
		Yes No	
		Yes No	

Depth of Carbonation *Attach additional documentation and micrographs as needed.*		
Core ID	*Depth of Carbonation*	*Comments*

Comments____

Figure II-8. Group 4: Summary of staining tests.

Analyst	Job ID
Date	Sample ID

Petrographic OM - Observations of the Concrete

Attach additional documentation and micrographs as needed.

Diagnostic Feature	*Selected Descriptors*	*Comments*
w/c in bulk?	specify method and value	
w/c at surface if different than bulk?	specify method and value	
Evidence of trapped bleed water?	water voids below horizontal aggregate faces	
Air-void structure at surface intact?	specify	
Calcium hydroxide depletion?	specify	
Sub parallel cracking or delamination at surface?	specify	
Paste density variations around the aggregates?	specify	
Cracking?	cracks through aggregates cracks around aggregates cracks in paste	
Secondary deposits?	yes no	
Location of secondary deposits?	air voids cracks through aggregates cracks around aggregates cracks in paste	
Identify deposits.	specify	
Lithological details of coarse aggregate?	specify	
Lithological details of fine aggregate?	specify	
Mineral admixtures?	present not present	
Identification	fly ash silica fume GGBF slag other	

Narrative Description of Petrography Results: ____________________

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Figure II-9. Group 5: Petrographic OM - observations of the concrete.

Analyst	Job ID
Date	Sample ID

Petrographic OM Observations - Alterations of the Aggregates

Attach additional documentation and micrographs as needed.

Coarse Aggregates

Diagnostic Feature	*Selected Descriptors*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution of aggregate?	yes no	
Reaction rims?	yes no	
Reaction products?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	
Identify reaction products	specify	

Fine Aggregates

Diagnostic Feature	*Selected Descriptors*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution of aggregate?	yes no	
Reaction rims?	yes no	
Reaction products?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	
Identify reaction products	specify	

Narrative Description of Alteration ____

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Figure II-10. Group 5: Petrographic OM observations - alterations of the aggregate.

Analyst	Job ID
Date	

SEM - General Conditions	
SEM or CSEM	Operating Pressure
Samples Conductive Coated (Y/N)	Coating Method/Thickness
Samples Dehydrated (Y/N)	Dehydration Method

SEM - Conditions for Quantitative Microanalysis	
SEM or CSEM	Operating Pressure
Accelerating Voltage	Beam Current
Working Distance	Standardless or full-quantitative (with standards) analysis?
Oxygen measured or determined by stoichiometry?	Analysis done by EDS or WDS?

SEM - Conditions for X-ray Mapping	
SEM or CSEM	Operating Pressure
Accelerating Voltage	Beam Current
Working Distance	Map Resolution
Map Dwell Time	Elements Mapped

Comments ____

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Figure II-11. Group 6: SEM - general conditions.

Analyst	Job ID
Date	Sample ID

Summary of Scanning Electron Microscope Analysis

- Summarize observations and hypotheses. Note micrograph numbers and spectrum identifications when referenced. Attach all data in a folder labeled with the core number and the title SEM Data.

Figure II-12. Group 6: Summary of scanning electron microscope tests.

Analyst	Job ID
Date	

Summary of Chemical Tests

Determination of w/c			
Specimen ID	Method Used	Measured w/c	Comments

Determination of Sulfate Concentration			
Specimen ID	Method Used	Sulfate Concentration	Comments

Determination of Chloride Concentration			
Specimen ID	Method Used	Chloride Concentration	Comments

Figure II-13. Group 7: Summary of chemical tests.

Analyst	Job ID
Date	

XRD - Analytical Conditions	
Type of specimen (e.g., powder, slab)	Specimen mounting method? (Side drifted, pressed pellet)
X-ray tube kV and mA	X-ray tube target material
Was a primary beam filter used and what type?	Monochromater used? (Y/N)
Divergence slit (specify mm or degrees)	Receiving slit (specify mm or degrees)
Scan range (degrees 2θ)	Scan rate (degrees/min)
Dwell time (seconds/step)	Step size (degrees/step)
Peaks identified by automatic or manual search (auto or manual)?	Background subtracted before analysis?

XRD - Results of Qualitative Analysis

- Summarize observations and hypotheses. List phases identified. Note diffraction pattern identifications when referenced. Attach all data in a folder labeled with the core number and the title Qualitative XRD Data.

Figure II-14. Group 8: XRD - analytical conditions and results of qualitative analysis.

Group 1: Core Receipt and Cataloging

Cores from a project will enter the laboratory accompanied by the coring log described in *Guideline I — Field Distress Survey, Sampling, and Sample Handling Procedures for Distressed Concrete Pavements* and each core will have a core identification label affixed to it. These should be carefully examined for consistency to ensure that improper labeling had not occurred in the field. Once verified, the information provided is entered in the laboratory log of PCC pavement cores provided in figure II-2. To simplify data recording if the core is not in one piece, isolate the three biggest or most interesting pieces as number 1 through 3 and combine the remainder as piece number 4. In these cases, the general position of the piece in the reconstructed core is also recorded in figure II-2. The top of the core is defined as the pavement wear surface. When an arrow is drawn on the top of the core to define the direction of traffic, the arrow points to the front of the core. The left and right sides of the core may be determined by orientating the core such that the observer is facing the tip of the arrow. The physical attributes (height, weight, diameter) of each core used to prepare specimens for laboratory analysis will be recorded in figure II-3. Note obvious physical defects and include photographs and a sketch of the core.

Once received and cataloged, specific cores of interest will be sampled and specimens will be prepared. If the core is intact, it can be sliced in one of two ways. The first approach is to slice the core lengthwise through the middle. In this case, one of the halves will be saved for later staining, chemistry laboratory analysis, or as an archival specimen. The other half can again be sliced to produce a 25- to 40-mm-thick slab for use in an OM evaluation. Thin sections can be made from interesting areas in the slab or from the remaining piece of the sectioned half. This type of approach is appropriate in evaluating the change in condition and characteristics of the concrete with depth. The second method is to slice the core perpendicular to its length. This approach is used to determine concrete condition or characteristics at selected depths.

It is likely that when advanced stages of deterioration are present, cores will not be extracted intact, but instead be obtained in a number of pieces. When possible, these pieces should be reassembled, and epoxy impregnated for strength. Slicing should be done in a manner consistent with the above recommendations (i.e., sliced lengthwise and parallel to the direction of traffic flow). This may not be possible in all cases due to the size and orientation of the fragments, and therefore the analyst will need to use discretion as to which pieces will be prepared

In this guideline, the general recommended approach is the first, in which the core will be sliced lengthwise and parallel to the direction of traffic flow. But, depending on the objective of the examination, one or both of the approaches may be appropriate. For example, the analyst may want to isolate the surface if studying scaling. Even if the first slice is made lengthwise, a perpendicular cut could be made subsequently to isolate the near surface zone. The use of the 150-mm-diameter cores allows more flexibility in preparing samples.

Slicing the core parallel to the direction of traffic flow assists in investigating degradation in the vicinity of joints and helps establish a consistent pattern for all cores obtained. As described in *Guideline I — Field Distress Survey, Sampling, and Handling Procedures*, the coring locations and number of cores obtained is dependent to some degree on the observed distress. If the distress is concentrated in the vicinity of joints or cracks, a total of four cores (A through D) will be obtained for each coring site. Five cores (A through E) will be obtained from each coring site if the MRD is not concentrated at the joints or cracks. Cores A, B, and C are being obtained specifically to investigate deterioration at or near joints or cracks. It is noted that Core A will contain a section of a dowel bar or reinforcing steel [in the case of continuously reinforced concrete pavement (CRCP)], which will be evaluated for corrosion or socketing of the concrete. Cores D and E are located in mid-slab locations and are taken in good concrete and distressed concrete, respectively. It is also likely that an additional core may be obtained from a given coring site to investigate a relevant feature. For example, if the field inspection team notes highly visible vibrator trails, they may obtain an additional core from such a feature in order to determine if segregation of inadequate air voids exists. For more information on the recommended coring procedure, see *Guideline I — Field Distress Survey, Sampling, and Handling Procedures*. A discussion of the various methods for preparing specimens is presented in appendix B.

Group 2: Visual Inspection

The visual inspection entails observations made of the specimen with the naked eye or a hand lens. The general condition of the concrete is assessed in accordance with the questions posed in figure II-4.

Many of the features observed during the initial visual examination will be examined more closely during the stereo OM evaluation. But the qualitative observations made during this inspection help to provide an overall assessment of the concrete as well as focus future investigations.

Group 3: Stereo Optical Microscope Examination

Of all the available microscopic methods, the stereo OM is clearly the first major analytical instrument to use when analyzing concrete. It provides a large depth of field, with sufficient magnification, to see most of the key features in concrete microstructure. When viewing a polished slab of concrete, higher magnifications are usable and key microstructural features of many components, including the aggregates, paste, air void structure, reaction products and cracks, are clearly visible. Often, a petrographer can diagnose MRD with this approach alone. Given the relatively low cost of this instrument, a stereo OM is a must for any laboratory attempting to diagnose MRD.

The most common types of concrete specimens observed in an optical microscope are broken pieces, polished slabs, and thin sections. Broken pieces require no sample preparation if examined by stereo OM. This is often very useful for examining fractures, aggregates, and filled voids. Polished slabs and thin sections require preparation time but provide for more detailed analysis of the concrete. Details regarding equipment, sample preparation, and methods for the stereo OM are provided in appendix B. Data collection forms are presented in figures II-5 through II-7.

Group 4: Staining Tests

Staining techniques can be effectively used to guide the entire laboratory analysis at an early stage. Generally speaking, staining can be carried out in any lab space and may very well be conducted in the petrographic laboratory if no chemistry lab space is available. The SHRP uranyl acetate test (Stark 1991) is one exception due to special requirements regarding chemical handling and the need for viewing under UV light.

Four existing staining methods are discussed in this guideline. They are the barium chloride potassium permanganate (BCPP) stain for sulfate minerals in concrete (Poole and Thomas 1975), the sodium cobaltinitrite/rhodamin B method for diagnosing alkali–silica reactivity (ASR) (Guthrie and Carey 1997), and the SHRP uranyl acetate method for identifying ASR in concrete (Stark 1991). More information on the SHRP uranyl acetate method is given in an appendix in ASTM C 856. An additional staining test uses phenolphthalein to stain uncarbonated cement paste a magenta color. Carbonated paste will not stain. The use of these stains is discussed in appendix B. Figure II-8 presents a data collection form for staining results.

Group 5: Petrographic Optical Microscope Examination

The petrographic OM is also a very useful instrument for evaluating microstructure and identifying the composition or mineralogical characteristics of phases within concrete. Using polarized light, the petrographer can identify subtle optical properties in different minerals that are characteristic and often unique to that mineral. By this method, a more complete picture of the concrete microstructure is obtained. The drawback to the petrographic OM is two-fold. First, there is often the need for preparing concrete thin sections which requires practice and experience. Some petrographers skip this step and analyze individual grains picked from the concrete using refractive index liquids. However, the second problem is that even without thin sections, the proper interpretation of petrographic OM data requires significant training and experience to identify all possible minerals. With practice, most lab personnel can learn to identify the principal minerals composing the microstructure of concrete using the petrographic microscope but it will take some time and effort. More details regarding equipment, sample preparation, and methods for the petrographic OM are provided in appendix B. Data collection forms are presented in figures II-9 and II-10.

Group 6: Scanning Electron Microscope Examination

Like the petrographic OM, the SEM can be used to identify the composition or mineralogical characteristics of phases within concrete. However, unlike the petrographic OM, the SEM can produce results that are easily interpreted by most lab personnel. As the phase of interest is ionized with an electron beam, the chemical analysis is determined by x-ray microanalysis using an energy dispersive (ED) or wavelength dispersive (WD) detector that measures the characteristic energy or wavelength of x-rays produced. The x-ray signal is used to produce elemental maps that identify common occurrences of elements. Also, a quantitative analysis can be produced that reports the elements present in the ionized phase and their respective concentrations. This last capability is most significant as it allows for accurate identification of mineral species, regardless of their crystallinity (i.e., the composition of amorphous reaction products are readily determined).

The CSEM is also capable of producing an electron image of the concrete showing the microstructural features and cracks. On first inspection, given the high magnification level of a conventional CSEM, it would seem ideally suited for studying cracks in concrete. Unfortunately, this is not the case. The instrument operates at a very low pressure (10^{-6} torr), which dehydrates the concrete when it is placed in the CSEM, potentially leading to significant cracking. The ESEM operates at a higher relative pressure (2-20 torr) and reduces this effect to a minimum. However, some cracking may still occur, depending upon operating conditions, and care should be exercised in interpreting micron size cracks seen in an ESEM image as signs of distress. Also, the ESEM is not ideally suited for x-ray microanalysis as the higher operating pressure relative to an CSEM results in more collisions between gas molecules and the exciting electron beam. This leads to a scattering of the incident beam and production of x-rays in phases not being analyzed. These "background" x-rays appear in the spectrum obtained and can lead to erroneous phase identification. This problem is minimized with a short working distance and low operating pressure. A compromise instrument is the low vacuum SEM (LVSEM), which operates at pressures ranging from 10^{-6} to 2 torr. It provides the analytical performance of a CSEM when operated at high vacuum but also allows for viewing of uncoated concrete with little to no desiccation cracking at low vacuum. Details regarding equipment, sample preparation, and methods for the SEM are provided in appendix B. The data collection forms for the SEM are presented in figures II-11 and II-12.

Group 7: Chemical Tests

The use of chemical tests in the diagnosis of MRD is one possible exception to the order of applying tests. In figure II-1, they are presented as a step following the stereo OM examination, but in reality they can be conducted at almost any point in the evaluation, and are often not performed at all. It is noted that many wet chemistry procedures require relatively large volumes of concrete, and thus they would likely be conducted on the half of the core put aside for this purpose as discussed previously. Also, some chemical analyses take many days to complete and work should begin immediately if the results are to be obtained in a timely manner. If wet chemistry laboratory tests are to be conducted they should be considered at an early stage of the analytical process for all these reasons.

The role of the chemistry laboratory in analyzing an MRD in concrete is not always clearly defined and whether the chemistry laboratory is used or not depends upon availability and cost. The initial cost of installing a full laboratory may be prohibitive for a SHA currently without this capability. Private laboratories are available and when the need arises, specimens can be sent out. However, many of the tests can easily be carried out in a conventional, well-ventilated laboratory without the need for sophisticated analysis equipment. A data collection form for chemical tests is presented in figure II-13.

Group 8: X-ray Diffraction Analysis

X-ray diffraction is not commonly applied to concrete, although in many cases it could assist in clarifying complicated problems. The fundamental data obtained in an XRD diffraction pattern is the identification of crystalline phases in concrete. Unfortunately the diffraction patterns from different phases of a multiphase specimen convolute together in an additive manner, creating an often-unrecognizable pattern. In this latter case, which is very common in concrete, diffraction patterns from phases low in concentration are masked. For this reason, conventional XRD analysis using a diffractometer is not useful for identifying minor components of the concrete in bulk samples. Often the analyst can "pick" materials from the concrete matrix and these small quantities are analyzed using XRD. Because the phase of interest was extracted from the matrix, interfering diffraction patterns from other phases are eliminated. However, the intensity of the pattern is reduced drastically. Also, the analyst must keep in mind that the extracted phase may actually be a mixture of two or more phases (e.g., ASR reaction product and calcium hydroxide). Data collection forms for XRD are presented in figure II-14.

Summary on Laboratory Testing

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.

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 AAR?” or “Is the air-void system adequate for the concrete service conditions?” After examining the concrete and noting all available information, the analyst can only make an educated judgment as to why the material failed. In some cases, there will be 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.

Description of the Data Interpretation Flowcharts

The purpose of this section is to outline the procedure for gathering and interpreting data for identifying MRD in concrete pavements. The proposed method uses the flowcharts and diagnostic features to guide the analyst toward diagnosis. The flowcharts present a systematic method for diagnosing MRD in concrete pavements. The analyst inspects the concrete using the methods described previously, being guided by the hierarchy of questions presented in the flowcharts. The responses to the questions presented in the flowcharts determines what analytical procedures will be performed. As the analyst moves through the flowcharts, there is the potential for more than one MRD being identified. The analyst keeps track of all possibilities identified by checking the appropriate response in the table at the top of the flowchart. Tables presenting the diagnostic features are then consulted to help isolate the most likely MRD(s).

Flowchart for Assessing the Likelihood of MRD

As discussed, figure I-14 is used to evaluate the likelihood of an MRD causing the observed distress. This flowchart, which should already have been completed, uses information gathered during field observations to determine if a laboratory evaluation is warranted. However, it is worthwhile for the analyst to ask the same questions and/or review the relevant data to understand the nature and scope of the distress. The questions asked are based upon visual inspection, with a positive answer to any question resulting in the diagnosis of a possible MRD necessitating further study.

Flowchart for Visual Inspection

Next, the analyst needs to perform a visual inspection of the concrete to assess its general properties. This may be done on the as-received cores, on broken pieces, and/or on polished slabs. The questions to be asked are presented in figure II-4. In most cases, visual inspection helps build a hypothesis of failure, but does not itself directly lead to the cause of distress. Most of the observations will be confirmed at a later stage using OM or SEM. However, there are some general conditions, with visual diagnostic features, known to contribute to, or directly cause, distress. The process to identify these conditions is illustrated in figure II-15 and is described as follows:

- Assess the mix volumetrics, examining the paste content, aggregate content, and apparent air content as compared to what was specified or what would be normal for that type of mix. SHAs, in general, work with a finite number of mix designs and an experienced analyst benefits from having observed cores of the same mix design from various pavements, simplifying this assessment. A mix with incorrect volumetrics may have been improperly proportioned.
- Check for consolidation problems. Poorly consolidated concrete is typically more permeable and as a result, is more susceptible to opportunistic MRD. Poor consolidation can be caused by poor placement, poor mix design, and/or improper proportioning.
- Check for segregation or parallelism of the aggregates. Both are symptoms of poor placement, poor mix design, and/or poor proportioning and may be significant contributors to the observed MRD.
- Check for scaling or subparallel cracking at the road surface. This is indicative of possible poor finishing and/or curing. The subparallel cracking may be very fine and only seen by use of a hand lens or stereo microscope. However, in some cases, these cracks are easily seen with the naked eye.
- For concrete with corroded reinforcing steel, if inadequate concrete cover (<50 mm) is provided over the steel, the observed cracking and spalling may be caused by the corroded steel, which was placed too close to the surface to be protected.
- Check for water voids that are trapped below flat or elongated coarse aggregates. This is indicative of excessive or trapped bleed water, which in turn can be a result of poor mix design, poor proportions, poor placement, and/or poor finishing.
- Check for surface cracking. Try to connect cracks to aggregates, noting aggregate type, and look for any sign of reaction such as reaction rims, reaction products, or cracked aggregates. Unfilled cracks are most probably shrinkage cracks. If shrinkage cracking is seen, poor finishing and/or curing is possibly responsible.
- Measure the depth of carbonation. Paste carbonation depth can be easily determined by using the phenolphthalein staining technique. Carbonated areas appear unstained and the depth of penetration can be easily measured.

Flowchart for Analysis of the Paste and Air

After completing the visual inspection, the concrete is examined with the stereo OM and/or staining techniques are performed. In this process, data on the general condition of the concrete are collected using figures II-5 and II-8. Having completed this analysis, the analyst can begin the process of assessing the condition of the paste and air system. Quantitative measurements of the air system parameters should be obtained using methods consistent with ASTM C 457 and the results entered in figure II-7. In figure II-16, the process for analyzing the paste and air fraction of the concrete is shown. To complete this flowchart it may be necessary to use the petrographic OM, SEM, chemical analysis, or XRD. The results of these analyses are recorded in figures II-7 through II-14, as appropriate. The process to evaluate the paste and air is illustrated in figure II-16 and is described as follows:

Possible Distress	Present		Additional Information
Error in mix proportioning	Yes	No	See recommended literature
Poor placement	Yes	No	See recommended literature
Poor finishing/curing	Yes	No	See recommended literature
Poor steel placement	Yes	No	See recommended literature
Carbonation at depths > 5-10 mm	Yes	No	See recommended literature

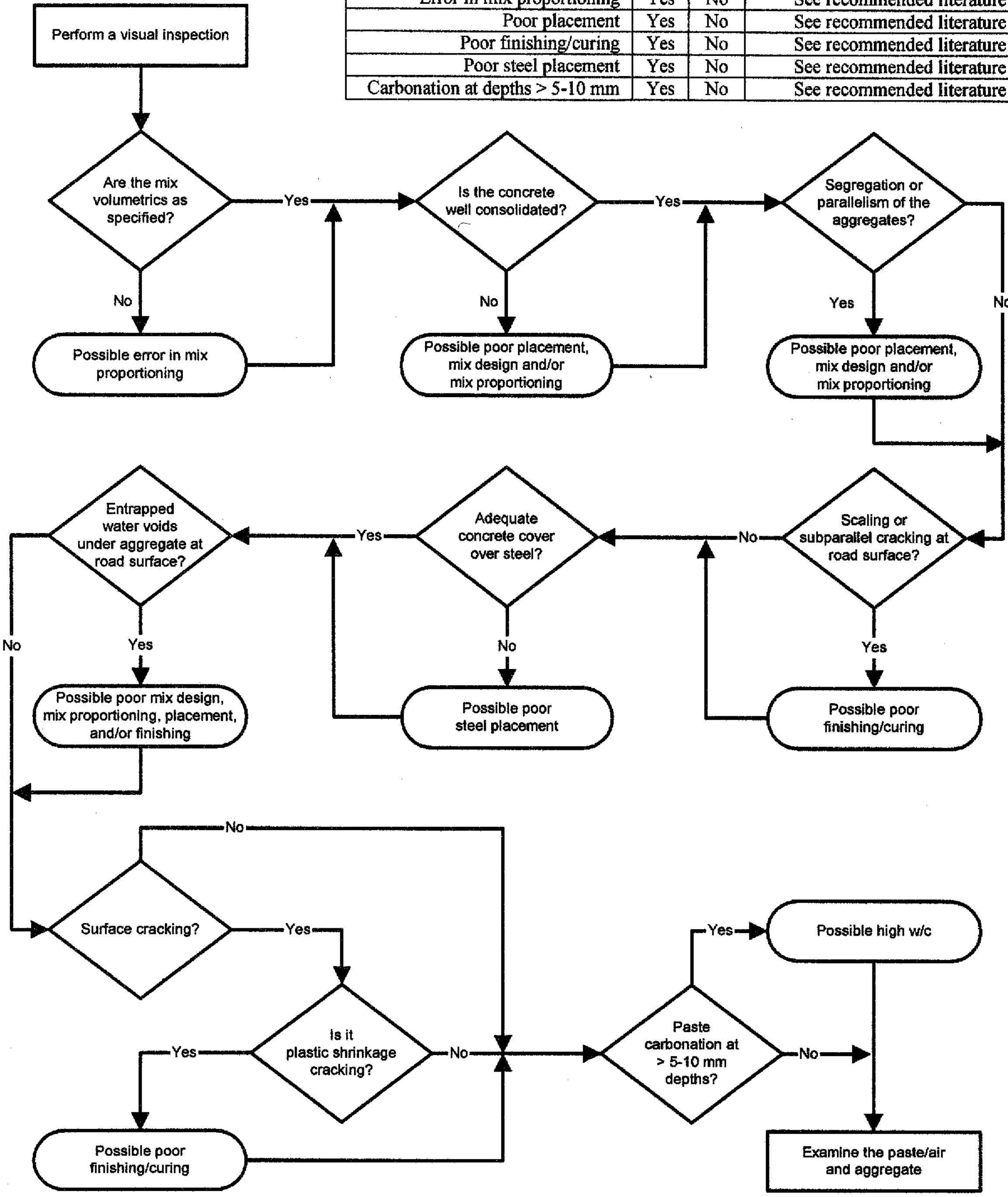


Figure II-15. Flowchart for assessing general concrete properties based on visual examination.

Possible Distress	Present		Additional Information
Shrinkage cracks or sample preparation cracks	Yes	No	See recommended literature
Corrosion of embedded steel	Yes	No	Table II-1
Paste freeze-thaw	Yes	No	Table II-2
Aggregate freeze-thaw	Yes	No	Table II-3
Sulfate attack	Yes	No	Table II-4
Deicer attack	Yes	No	Table II-5
Infilling material	Yes	No	Figure II-17

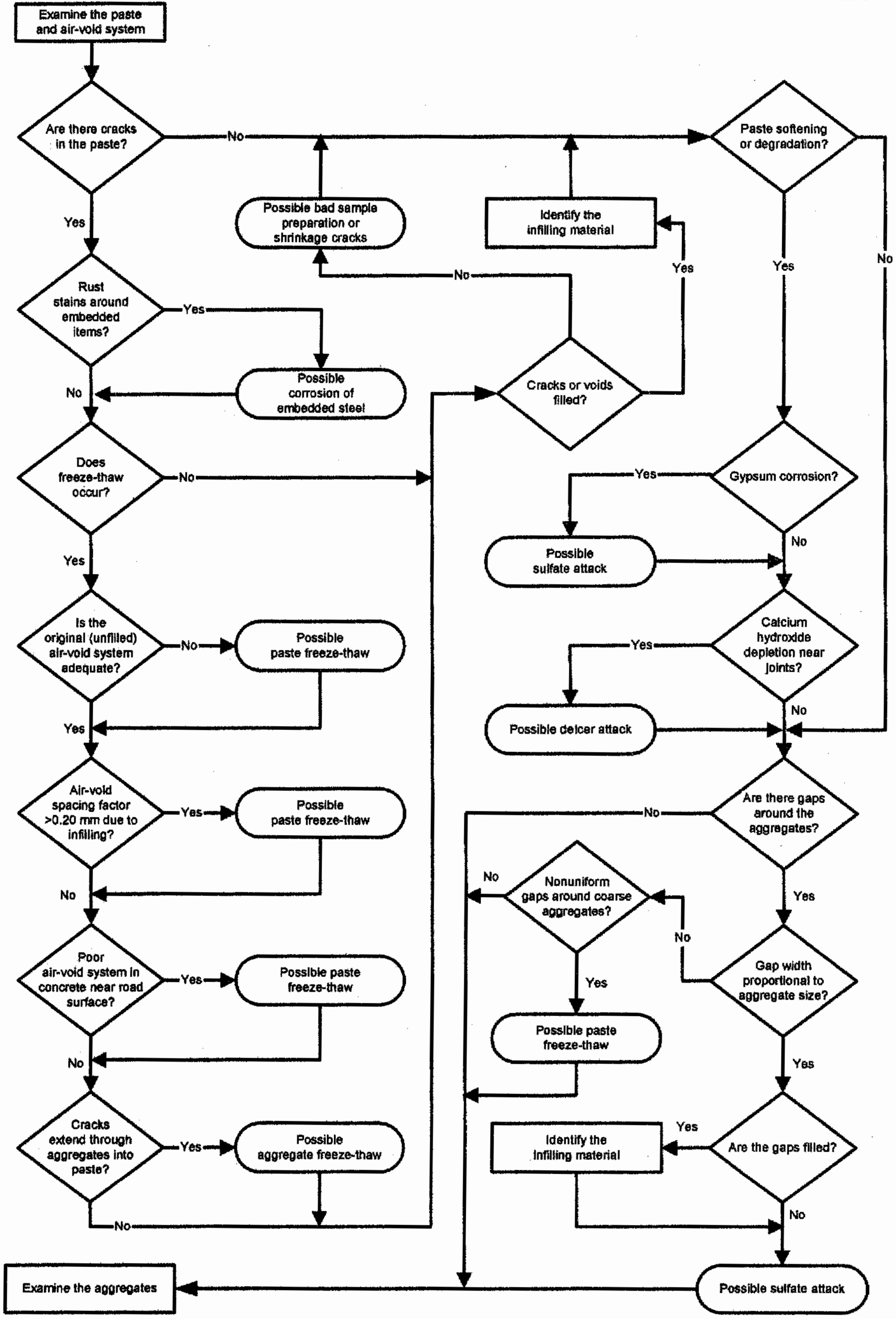
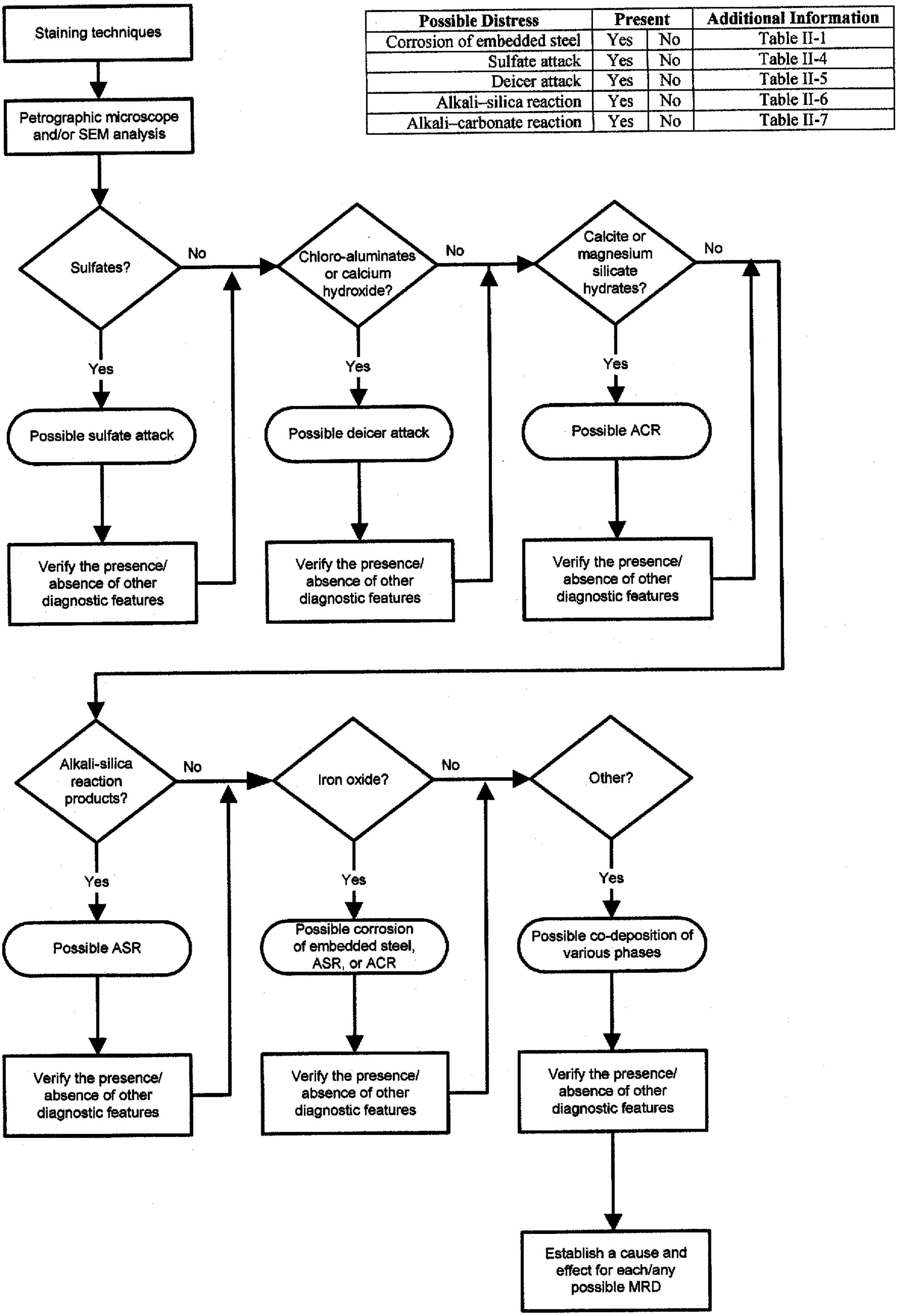


Figure II-16. Flowchart for assessing the condition of the concrete paste and air.

- Any cracks resulting from distress may only be apparent in selected cores (e.g. cores A and B from near the joint/crack).
- If paste cracking is present and there are rust stains around the embedded steel, the steel is corroded and probably contributing to the distress. See table II-1 for diagnostic features.
- If paste cracking is present, and the service condition for the concrete is classified freeze-thaw, determine the air-void system parameters using methods consistent with ASTM C 457. If the air system is inadequate, paste freeze-thaw damage is possible. See table II-2 for diagnostic features.
- If the voids are infilled with secondary deposits, it may be necessary to determine the effect the infilling has on the air-void system parameters. If the infilling material is counted as an air void, the linear traverse or point counting analyses will yield the original entrained air system. If the infilling is counted as paste, then the existing air-void system will be measured with an increase in the Powers spacing factor. If this increase in spacing factor changes a satisfactory original air content to unsatisfactory, then the infilled air-void system may not be performing as required. For more information on paste freeze-thaw distress, see table II-2 for diagnostic features. As part of the ASTM C 457 analysis, the volume fraction of all components can be measured to determine the mix volumetrics.
- Check for a reduction of hardened air content at the road surface. This is indicative of overworking during finishing, possibly leading to scaling that can be exacerbated by deicer application. For more information on paste freeze-thaw distress and deicer attack, see table II-2 and II-3 for diagnostic features, respectively.
- Cracks in the paste extending through the aggregates may indicate aggregate freeze-thaw distress. The analyst should consult table II-3 for more diagnostic features of aggregate freeze-thaw distress.
- For cracks or air voids that are filled with secondary deposits, determine the composition of the infilling material as described in figure II-17. Unfilled cracks are typically either shrinkage cracks or the result of sample preparation. Assess the paste quality or hardness. Softening or crumbling of the paste can be caused by gypsum corrosion (sulfate attack) or deicer attack.
- Gypsum corrosion results when sulfate-laden water contacts concrete, converting the cement paste to a noncementitious "mush." The sulfate source may be external or internal to the concrete. For diagnostic features on sulfate attack, see table II-4.
- Deicer attack manifests itself in many ways. Some common symptoms are depletion of calcium hydroxide near joints with possible redeposition in cracks and voids. Another common indicator of deicer attack is the presence of chloroaluminates infilling voids or cracks. Deicer applications also exacerbate paste freeze-thaw damage. For more information on paste freeze-thaw damage and deicer distress, see tables II-2 and II-5, respectively.
- Check for gaps around the aggregates. If the gap width is proportional to the aggregate size, then the gaps may be the result of paste expansion, a common symptom of sulfate attack. An additional diagnostic feature of sulfate attack is sulfate phases deposited in the gaps and cracks, so identify any infilling material as described in figure II-17. For diagnostic features on sulfate attack, see table II-4. If the gaps are not proportional and are only around coarse aggregate, it may be aggregate-freeze thaw distress. See table II-3 for more information.

Flowchart for Analysis of the Aggregate Structure

After completing the visual inspection, the concrete is examined with the stereo OM. In this process, general data on the condition of the aggregates are recorded using tables II-5 and II-8.



Possible Distress	Present		Additional Information
Corrosion of embedded steel	Yes	No	Table II-1
Sulfate attack	Yes	No	Table II-4
Deicer attack	Yes	No	Table II-5
Alkali–silica reaction	Yes	No	Table II-6
Alkali–carbonate reaction	Yes	No	Table II-7

Figure II-17. Flowchart for identifying infilling materials in cracks and voids.

Having completed these analyses, the analyst can begin the process of assessing the condition of the aggregate structure. Aggregate quality is fundamental to concrete performance and any degradation of the aggregates should be closely scrutinized. To complete this process it may be necessary to use the petrographic OM, SEM, chemical analysis, and/or XRD. Record the results of these analyses in figures II-7 through II-14, as appropriate. In figure II-18, the process for examining the aggregate fraction of the concrete is presented.

- Any cracks resulting from distress may only be apparent in selected cores (e.g., cores A and B from near the joint/crack)
- Check for aggregate cracking. For both the coarse and fine aggregate, cracks in nonreactive aggregates that do not extend into the paste are most probably either a pre-existing defect in the aggregate or the result of poor sample preparation. The latter can easily be observed as the cracks will not be filled with secondary deposits, except for possibly polishing media. Epoxy impregnation also assists in making the distinction as cracks not filled with epoxy are certainly from sample preparation, assuming complete epoxy impregnation of the specimen. Aggregate cracks that extend into the paste, or are filled with secondary deposits, may be the result of aggregate freeze-thaw (see table II-3 for more information), ASR (see table II-6 for more information), or alkali–carbonate reactivity (ACR) (see table II-7 for more information).
- If the cracks are filled with secondary deposits, identify the infilling material as described in figure II-17.
- If the concrete is from a freeze-thaw environment, examine the coarse aggregate (>12.5mm) for evidence of aggregate freeze-thaw distress. Aggregate freeze-thaw susceptibility is commonly identified by reviewing construction and performance records or by petrographic examination of the aggregate. See table II-3 for diagnostic features on aggregate freeze-thaw.
- Check for reaction rims on aggregates. Visible rims with no evidence of dissolution, such as secondary infilling of cracks and voids, are probably the result of natural weathering of the aggregates. Reaction rims may be symptomatic of ASR (see table II-6 for more information) or ACR (see table II-7 for more information).
- If secondary deposits in cracks and voids are seen, identify the infilling material as described in figure II-17.
- Check for evidence of aggregate softening. This is often seen at the center of an aggregate and is indicative of dissolution of the aggregate. It generally results in an overall weakening of the aggregate and severe cracking results as the aggregate dissolution produces expansive reaction products. If the aggregate type is carbonate, then suspect ACR (see table II-7 for more information). Otherwise, suspect ASR (see table II-6 for more information). Note that some carbonate aggregates contain significant siliceous phases that may undergo ASR.

Possible Distress	Present		Additional Information
Natural cracking of aggregate	Yes	No	See recommended literature
Sample preparation cracks	Yes	No	See recommended literature
Aggregate freeze-thaw	Yes	No	Table II-3
Natural weathering of aggregates	Yes	No	See recommended literature
Alkali–silica reaction	Yes	No	Table II-6
Alkali–carbonate reaction	Yes	No	Table II-7
Infilling material	Yes	No	Figure II-17

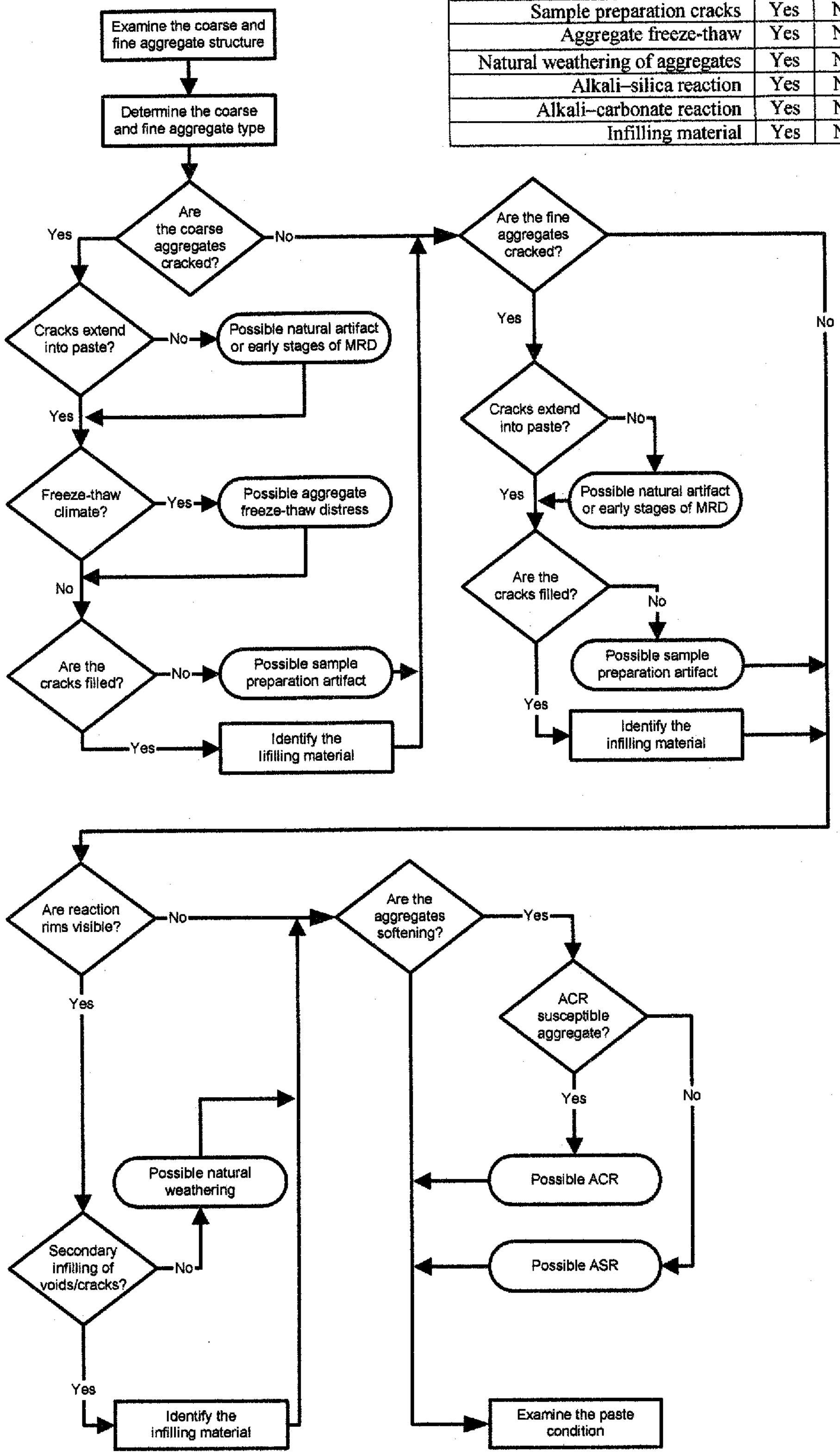


Figure II-18. Flowchart for assessing the condition of the concrete aggregates.

Flowchart for Analysis of the Secondary Deposits

In figure II-17, the procedure for identifying infilling material is presented. The identification is accomplished by using staining techniques, petrography, and/or x-ray microanalysis using the scanning electron microscope. The identification is fairly straightforward with staining techniques yielding a yes/no answer and x-ray microanalysis yielding a full chemical analysis that can be used to identify the specific material. Optical petrography can also be used, but it is more difficult than the other methods as the optical properties are more difficult to discern. ASTM C 856 includes a complete table of common minerals found in PCC and their optical properties. Regardless of the composition of the infilling material, the analyst must do two things to confirm the MRD type based upon identification of the infilling material.

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

Use of the Diagnostic Feature Tables

Tables II-1 through II-7 summarize the principal, common, diagnostic features for each MRD, as characterized by laboratory methods and as seen during field evaluation. Included in these tables are the diagnostic features, methods of characterization, and specific comments relative to either the observed characteristic or the test method. The tables are intended to serve as a quick reference during the diagnosis of MRD by assisting the analyst in identifying other diagnostic features to help confirm the presence of a particular MRD. The tables also serve to tie together the results of visual inspection performed in the field, with laboratory inspection and diagnosis.

While performing the laboratory analysis, the analyst will be guided by the flowcharts presented in figures II-15 through II-18. The flowcharts will be asking yes/no type questions about laboratory observations that are not always definitive. Usually, each question will focus on one diagnostic feature, the answer of which cannot be used alone to positively identify a specific MRD as the cause of distress. To confirm the identification, other diagnostic features need to be investigated and confirmed. At this point, the analyst should use the diagnostic tables presented in tables II-1 through II-7 to see what he/she should look for to help confirm the suspected MRD. It may be possible to perform the confirmation immediately, but in many cases, different samples must be made or techniques employed (e.g., thin sections, petrographic OM, etc.). In the latter case, it is recommended that the analyst proceed completely through the flowcharts once before going back to confirm specific MRD types. As other possible MRD types are identified, consult the diagnostic tables to determine other possible confirming diagnostic features. Note on the flowcharts which MRD types have been indicated as possible by circling yes or no in the table included at the top of each flowchart.

Table II-1. Diagnostic features of corrosion of embedded steel.

Diagnostic Feature	Method of Characterization	Comments
Spalling and delamination of concrete over reinforcing steel	Field Evaluation Visual Inspection	Visual inspections can be used to readily identify areas affected by corrosion of embedded steel. It is characterized by rusting steel at the bottom of the spalled out area and rust stains on the loose pieces.
Visible corrosion products	Field Evaluation Visual Inspection Stereo OM Petrographic OM	The "rust" seen may contain crystalline magnetite but is primarily amorphous.

Continuing through the flowcharts before going back to confirm the first MRD identified accomplishes two things. First, it helps remove the tunnel vision of looking at only one possible MRD. Second, moving completely through the flowcharts will identify questions that require additional techniques to answer. Overall, this process helps to identify 1) what questions need to be answered, 2) what techniques will be required, and 3) what samples will be needed. After completing the required analyses, the analysts should go through the flowcharts one more time, isolating the most probable MRD(s).

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

SUMMARY OF DATA ANALYSIS AND INTERPRETATION

This second guideline presents laboratory procedures and an approach to diagnosing materials-related distress in concrete pavements. It is emphasized that trained, skilled personnel are needed to execute the recommended laboratory procedures, carry out the data collection function, and draw conclusions from the data collected. It is also emphasized that these guidelines provide only a framework for analysis and interpretation, and that additional sources of information need to be consulted in the course of an evaluation. And finally, it is hoped that the use of these guidelines will enhance communication between the analyst (typically a petrographer) and the field/design engineer, so that each can develop a better understanding of the other's needs and expectations regarding the strengths and limitations of such an investigation.

Table II-2. Diagnostic features of paste freeze-thaw damage.

Diagnostic Feature	Method of Characterization	Comments
Surface scaling or subparallel cracking	Field evaluation Visual	Look for loss of paste at road surface, exposed coarse aggregate, and/or scaling. Can be isolated to surface or through slab depth.
	Stereo OM Petrographic OM	Look for delamination/ subparallel cracking at surface or evidence of an overworked surface such as decreased air content at the surface.
Inadequate air-void system	Stereo OM	Measure air-void parameters consistent with ASTM Method C 457. Typical parameters for good concrete are as follows: Spacing Factor ($\overline{L}$) = 0.1 - 0.2 mm Maximum Spacing ($\overline{L}$) = 0.2 mm Typical Specific Surface (α) = 23.6 - 43.3 mm^{-1} Paste/Air Ratio - 4 - 10 The air-void size distribution should also be noted as different size distributions can yield similar values of $\overline{L}$ and α.
Secondary deposits filling air voids	Staining	Deposits stained cannot be analyzed by any other method to determine their composition.
	Stereo OM	Deposits result as water freely moves through the distressed paste. In extreme cases, the air-void system may be further compromised when significant numbers of voids are filled with secondary deposits. Common deposits include calcium hydroxide, calcium carbonate, ASR reaction products, and various sulfates including ettringite.
	Petrographic OM	Secondary deposits are commonly identified using the petrographic OM.
	SEM	The SEM operated at high vacuum is very useful for determining the composition of secondary deposits. Direct output of phase composition allows for absolute identification.
Microcracking around aggregates	Stereo OM Petrographic OM	Cracking will be in the paste. If cracks pass through aggregates, check table II-3 for coarse aggregate freeze-thaw, table II-6 for ASR, and table II-7 for ACR. Cracks will fill with secondary deposits.
	LVSEM	Severe cracking in the paste occurs if PCC is observed in a CSEM. Hydration products and ASR reaction products dehydrate in the high vacuum in the CSEM.

Table II-3. Diagnostic features of aggregate freeze-thaw deterioration.

Diagnostic Feature	Method of Characterization	Comments
Cracking near joints/cracks Staining/Spalling	Field evaluation	Has a very characteristic cracking pattern concentrated at corners, joints, and cracks (SHRP 1993). Increased permeability results from the cracking. Calcium hydroxide is leached and re-deposits on surface where it carbonates.
Cracks through nonreactive coarse aggregates	Visual inspection Stereo OM Petrographic OM SEM	Cracks through nonreactive coarse aggregates are very typical of D-cracking. Be very careful to completely rule out alkali–aggregate reaction. See tables II-6 and II-7 for diagnostic features of ASR or ACR.
Nonuniform gaps around coarse aggregates	Visual inspection Stereo OM	Gaps between the aggregate and paste form. These gaps may result from the dissolution of calcium hydroxide at the aggregate/paste interface or coarse aggregate dilation due to freezing. Subsequent redeposition of calcium hydroxide or calcite may occur in the cracks. Check for sulfate or ASR reaction products in cracks surrounding aggregates. A negative result helps to confirm aggregate freeze-thaw.
Known freeze-thaw susceptible aggregate Large top size aggregate	Records review Visual inspection Stereo OM SEM	Check aggregate sources for known freeze-thaw performance Aggregate freeze-thaw is more common in large aggregates (> 38 mm) and rare in aggregates smaller than 12.5 mm.
Poor void structure in the aggregate	Petrographic OM CSEM LVSEM	As a percentage of the total aggregate void space, excessive amounts of voids in the aggregate with diameters less than 5 microns is thought to be detrimental to aggregate freeze-thaw resistance.

Table II-4. Diagnostic features of sulfate attack.

Diagnostic Feature	Method of Characterization	Comments
Map cracking	Field evaluation	Paste expansion commonly results in map cracking over entire surface. In some cases, it is isolated to joints/cracks.
Deteriorated paste		Sulfate attack may result in paste "crumbling," commonly at joints. Loose aggregate observed in resulting void.
External source of sulfur only	Soil analysis Deicer analysis	Identify a sole source of sulfate that is external to the concrete to confirm external sulfate attack. Having both external and internal sources confounds the diagnosis.
Internal source of sulfur only	Records review SEM XRD	Identify a sole source of sulfate that is internal to the concrete to confirm internal sulfate attack. Having both external and internal sources confounds the diagnosis.
Paste expansion	Stereo OM Petrographic OM SEM	Expansion occurs, usually over a large area. Gaps form around aggregates with the gap width proportional to the aggregate diameter.
Significant sulfate deposits in cracks and voids	Staining Stereo OM	Stained deposits or stained paste cannot be accurately analyzed by SEM to determine their composition. A common sulfate deposit is ettringite. This is commonly recognized by acicular needle-like crystals infilling voids and cracks.
	Petrographic OM	Common sulfate deposits can be identified. Mixtures with other phases may be more difficult to identify.
	SEM	All deposits are readily identified using the SEM in high vacuum mode. Co-deposition with other phases may be more closely studied.
Significant sulfate deposits in the cement paste	Petrographic OM	Fluorescent dye epoxy impregnation assists in identifying microcracks in the paste. Cracks unfilled with epoxy should be assumed were created during sample polishing.
	LVSEM/ESEM	Cracks resulting from sulfate expansion can be viewed in an LVSEM/ESEM but caution should be exercised in identification of micron scale microcracks. Even in an ESEM, some dehydration does occur, leading to possible cracking. Cracks unfilled with epoxy should be assumed were created in the SEM or during sample polishing. If viewing an unimpregnated specimen, cracks unfilled with secondary deposits should be assumed were created in the SEM or during sample preparation.
Microcracking	Stereo OM Petrographic OM	For filled cracks, the cracks may have been present from other distress and secondary deposits formed in the cracks. Fluorescent dye epoxy impregnation greatly improves the identification of microcracks in the paste. Cracks not filled with epoxy are probably artifacts of sample preparation.
	SEM	A characteristic spectrum for dehydrated ettringite has approximate element ratios of 1:2:4 (Al:S:Ca) by weight.

Table II-5. Diagnostic features for deicer scaling/deterioration.

Diagnostic Feature	Method of Characterization	Comments
Staining at joints or cracks	Field evaluation	Staining results from calcium hydroxide depletion and subsequent carbonation at surface.
Scaling or crazing of slab surface	Field evaluation Visual inspection Stereo OM	Common visual diagnostic feature. Similar and possibly related to paste freeze-thaw damage. See table II-2 for more on paste freeze-thaw damage.
Calcium hydroxide depletion near joints	Stereo OM Petrographic OM SEM	Calcium hydroxide (CH) is most soluble near the freezing point of water. Cyclic freezing and thawing from repeated deicer applications can accelerate the dissolution of CH near joints/cracks.
Secondary deposits of chloroaluminates	Petrographic OM SEM	Chloride ions released from dissolved salts can form these phases with aluminate phases in the paste.

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

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

Table II-6. Diagnostic features of ASR.

Diagnostic Feature	Method of Characterization	Comments
Map cracking with or without exudate Evidence of pavement expansion	Visual inspection	ASR is characterized by widespread map cracking. Can be more severe at joints and may be preferentially oriented perpendicular to the direction of least restraint (e.g., in pavement slabs, longitudinal cracks often predominate). Exudate common but not always observed. Evidence of joint closing or shoving of shoulder or fixed structures are possible indicators of expansion.
ASR reaction product in cracks and voids	Stereo OM Petrographic OM	A glassy clear to white amorphous reaction product resulting from an alkali–silica reaction. ASR reaction product is found within reacted particles, cracks, and air voids. The presence of ASR reaction product alone does not indicate ASR distress, as it must be of sufficient volume and composition to cause deleterious expansion.
	SEM	ASR reaction product can be chemically characterized with the SEM operating at a high vacuum. Primarily high alkali (low calcium) ASR reaction products are expansive.
Reaction rims on aggregates	Visual inspection Stereo OM Petrographic OM SEM	Reaction rims are often seen on most reactive aggregate. Reaction rims are common on aggregates that are undergoing ASR. Good gravel aggregates can exhibit rims that appear similar to ASR reaction rims. These are typically the result of weathering. Reaction products present help confirm ASR.
Aggregate known to be reactive	Records review	Check to see if the aggregates used were from a source that is known to be reactive.
Microcracking radiating from reacted cracked aggregates Softening of the aggregate	Visual inspection Stereo OM Petrographic OM LVSEM/ESEM	Reacted aggregates may break down internally and often partially dissolve. As the aggregate degrades, the ASR reaction product produced may be expansive and cause cracking to occur. The cracks are within the periphery of the aggregate but around the center. Often the cracks will narrow from the center of the aggregate out. Coarse and fine aggregates can both cause ASR distress. Common reactive aggregates are composed of or include chert, flint, siliceous shale, strained quartz, and porous volcanic glasses.

Table II-7. Diagnostic features of ACR.

Diagnostic Feature	Method of Characterization	Comments
Map Cracking with or without exudate Evidence of expansion	Visual inspection	ACR is characterized by widespread map cracking. Can be more severe at joints. Exudate common but not always observed. Evidence of joint closing or shoving are possible indicators of expansion.
Cracks radiating from the coarse carbonate aggregate into the paste	Visual inspection Stereo OM Petrographic OM LVSEM	Deterioration from ACR results from the expansion of the aggregate that causes cracks in the aggregate, which propagate into the paste. The expansion is the result of reaction products produced in the dedolomization reaction.
Aggregate known to be reactive	Records review	Check to see if the aggregates used were from a source that is known to be reactive.
Characteristic texture of ACR reactive aggregates	Petrographic OM	Most ACR reactive aggregates have a characteristic texture. ASTM C 856 states the basic texture as being relatively larger rhombic dolomite crystals in a fine-grained calcite matrix with clay and silt-sized quartz. Substantial amounts of both dolomite and calcite are present. Other textures have been reported as reacting, with a common thread being soluble magnesium phases that react to form expansive products.
	CSEM LVSEM	Calcium and magnesium silicate hydrates are common reaction products.

GUIDELINE III — TREATMENT, REHABILITATION, AND PREVENTION OF MATERIALS-RELATED DISTRESS IN CONCRETE PAVEMENTS

This chapter presents the third and final guideline prepared as part of FHWA Project No. DTFH61-96-C-00073 entitled *Detection, Analysis, and Treatment of Materials-Related Distress in Concrete Pavements.* It consists of two major sections. The first section addresses treatment and rehabilitation of materials-related distress (MRD) affected pavements and the second section discusses the prevention of MRD in new concrete pavement construction. In the case of the former, it is assumed that the first two guidelines were used to evaluate the pavement under consideration. The latter section assumes that the State highway agency (SHA) has conducted a thorough investigation to determine the types and extent of MRD commonly affecting their pavements, and wants to prevent similar occurrences in new construction.

TREATMENT AND REHABILITATION OF MRD-AFFECTED PAVEMENTS

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

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

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

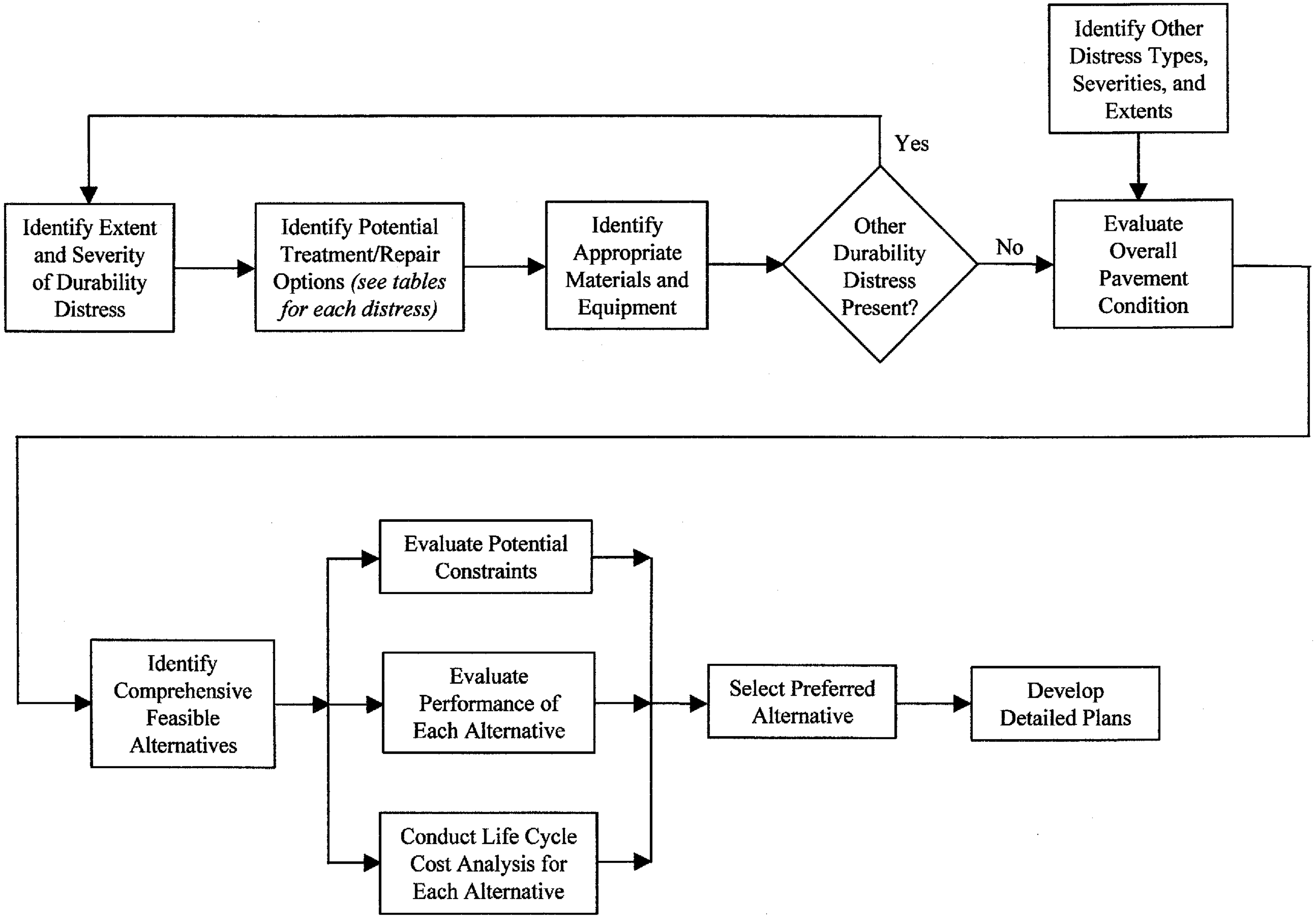


Figure III-1. Flowchart for selecting preferred treatment and rehabilitation options.

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 in this guideline under separate headings.

Available treatment methods include chemical treatments, joint and crack sealing, crack filling, surface sealing, and retrofitted drainage. Drying and restraint are two other treatment methods that show promise in some applications, but are not practical for pavement applications, and are thus not discussed. 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 durability problem in design and thereby prevent the distress from ever occurring. This is discussed in the last section of this guideline. 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 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 salts is an example of a chemical treatment that arrests an existing reaction. Lithium salts prevent abnormal expansion due to alkali–silica reactivity (ASR). 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 durability-related distress types. As a result, these distress types are often observed to be much more severe at joints and cracks, where moisture can penetrate the 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 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 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.

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).

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 guidelines, broken out for each particular MRD type, 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.

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 causes 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 move within the underlying layers to the edgedrains, 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 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 a smaller top aggregate size 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 III-1 provides some basic guidelines for selecting feasible repair alternatives based on the extent and severity of deterioration observed during the field data collection process described in guideline I. These should be viewed as rough guidelines for selecting potential feasible alternatives. Using the data recorded in Guideline I in figures I-8 through I-11 and the photo log, the severity of the distress can be assessed using the following guidance:

- 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

Table III-1. Selection of feasible alternatives to address freeze-thaw deterioration of aggregate.

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.

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, that must be addressed as part of the overall selection process.

Freeze-Thaw Deterioration of Cement Paste

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. Thus, as with freeze-thaw deterioration of aggregate, 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 most of the United States. 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. 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 extent 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 III-2. The selection of feasible alternatives depends on the extent and severity of distress observed during the field data collection process described in guideline I. As described in table I-1, paste freeze-thaw deterioration is most often manifest as map cracking, scaling, and/or spalling. Using the data recorded in figures I-8 through I-11 and the photo log, 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.
- 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 can not 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.

Table III-2. Selection of feasible alternatives to address freeze-thaw deterioration of cement paste.

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.

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.

Deicer Scaling/Deterioration

Deicer scaling/deterioration is caused by the repeated application of deicing chemicals, which accentuate many of the same stresses generated by cyclic freezing and thawing. High thermal strains produced when the deicing chemical melts the ice and increased ozmotic pressures are also thought to contribute to the problem. The problem is more likely on concrete pavements that have been overvibrated or improperly finished, creating a weak layer of cement paste over the surface (Mindess and Young 1981). Deicers also contribute to the dissolution of some hydrated paste components that are more soluble at temperatures at or below freezing.

Available Treatment Methods for Deicer Scaling/Deterioration

The application of deicing chemicals is a necessity in cold climates to prevent accidents. Elimination of the destructive component of the deterioration is therefore not an option. Elimination of freezing temperatures is also not possible. Treatment methods must therefore focus on preventing direct exposure between the concrete and the deicing chemicals.

The only feasible treatment method for deicer scaling is the application of a surface sealer. Surface sealers reduce the ingress of moisture and the chloride ions into the pavement without disrupting the frictional characteristics of the pavement. This method should only be used for pavements exhibiting limited amounts of low-severity scaling.

Available Rehabilitation Methods for Deicer Scaling/Deterioration

Deicer scaling is limited to exposed surfaces, mainly along the pavement surface. Diamond grinding can effectively remove the deteriorated concrete from the surface and provide a smooth-riding surface. It is most cost effective when the deterioration is widely distributed. Although diamond grinding removes the existing deterioration, it also exposes another layer of concrete to the deicing chemicals. If the underlying concrete is of higher quality because the surface was compromised due to poor finishing during construction (overfinishing or excess water on the surface during construction), this is a satisfactory long-term alternative. If on the other hand, the mass of concrete has a poor air-void distribution, deicer scaling will likely occur in the newly exposed surface.

Because scaling is confined to the surface, partial-depth repairs and full slab inlays can be an effective rehabilitation method. To be cost effective, the number and size of the repair areas should be limited. Small depressions and poorly finished areas are examples of good candidates for partial-depth repairs. If the area to be repaired is large or if multiple partial-depth repairs are required, full-depth repairs or slab replacements are more cost effective.

Other feasible alternatives are placement of an overlay, recycling, and reconstruction. These alternatives all address the entire pavement area. Overlays are likely to last longer on pavements exhibiting deicer scaling as compared to other MRD if the deterioration is limited to the surface because of poor finishing. Milling of the deteriorated surface before placing the overlay will help improve the support and bonding conditions. Because the distress is limited to the surface, recycling and reconstruction of the pavement will not be as cost effective as with other MRD types that effect the concrete through the entire slab depth.

Selection Guidelines for Deicer Scaling/Deterioration

Table III-3 provides a list of feasible alternatives and some general guidelines for selecting the preferred alternative to address scaling due to attack from deicing chemicals. The selection is based on the severity and extent of distress observed during the field data collection process described in guideline I. As described in table I-1, deicer scaling/distress is most often manifest as scaling or crazing with possible staining of the surface. Using the data recorded in figures I-8 through I-11 and the photo log, the severity of the distress can be assessed using the following guidance:

- Low severity is characterized by 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. Some staining in vicinity of joints might be observed.
- High severity is characterized by extensive scaling having a depth in excess of 12 mm. Coarse aggregate is exposed and in some cases missing. Patching and staining may be evident in affected area.

For low-severity scaling, the most cost-effective option depends on the extent and expected progression of scaling. For the most part, sealing the pavement with a silane sealer is the best method. This method may prevent further deterioration or prevent the development of scaling in other areas. For scaling that is limited to isolated areas and appears to be the result of improper finishing or standing water in depressions (i.e., scaling is not likely to become widespread), partial-depth patching and/or inlays can be effective alternatives.

The same guidelines for partial-depth repairs apply to moderate-severity deicer scaling if it is isolated and not believed to be progressive. Otherwise, the recommended alternative, especially when the deterioration is extensive, is to diamond grind the concrete surface to remove all deteriorated areas. After diamond grinding, consideration should also be given to the application of a surface sealer to delay the recurrence of scaling.

For high-severity scaling due to deicing chemicals, diamond grinding may not be effective because it only removes about 3 mm of concrete from the surface. For pavements with isolated areas of high-severity scaling, partial-depth repairs can be used. For larger areas, full-depth repairs, inlays, or slab replacement may be more cost-effective. If the deterioration is too widespread to repair isolated areas, the recommendation is to construct an AC overlay or bonded concrete overlay. Milling of the existing concrete surface to remove scaled areas is suggested to improve the bonding and support conditions. Recycling and reconstruction are also options but are unlikely to be cost effective except in extreme circumstances.

Table III-3. Selection of feasible alternatives to address deicer scaling/deterioration.

Severity	Extent	Feasible Alternatives	Comments
Low	Isolated areas	Seal the pavement Partial-depth patches	Limit the penetration of moisture and chloride ions.
	Entire slab	Seal the pavement Inlays[1]	
Moderate	Isolated areas	Seal the pavement Diamond grinding Partial-depth patches[1] Full-depth patches	Distress must be limited to upper one-third for partial-depth repairs to be feasible.
	Entire slab	Diamond grinding Inlays[1] Slab replacement Overlay	Diamond grinding (to remove scaling) can restore smoothness but will not prevent further development.
High	Isolated areas	Partial-depth repairs[1] Full-depth repairs Inlays[1] Slab replacement	Deterioration is too severe for treatment; deteriorated areas must be removed and replaced.
	Entire slab	Overlay (with milling) 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.

Alkali–Silica Reactivity

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 also 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 salts. Lithium salts 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 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 III-4 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 and low-lying areas (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. Using the data recorded in figures I-8 through I-11 and the photo log, 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 determined to be ineffective, then the best option is to let the pavement live out its life and then reconstruct the pavement.

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.

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 salts may also be effective. And although an AC overlay does not directly address the problem, it can be an effective method of improving serviceability.

Table III-4. Selection of feasible alternatives to address ASR.

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.

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 special considerations, the use of crushed concrete from pavements with ASR can be reused successfully.

Alkali–Carbonate Reactivity

Alkali-carbonate reactivity (ACR) involves a reaction between the alkalis in the cement and certain carbonate aggregates. The reaction is complex, and not well understood, but it is thought to involve a process called dedolomitization, in which dolomite is converted into calcium carbonate and magnesium hydroxide. The reaction produces expansive pressures that result in map-like cracking on the pavement surface and possibly spalls and blowups.

Available Treatment Methods for Alkali–Carbonate Reactivity

The deterioration due to ACR typically occurs throughout the entire pavement area, so treatments must address the entire pavement. No chemical treatments are currently available. The expansion does require the availability of moisture, and some research suggests that the presence of deicing salts exacerbates the distress. Consequently, the application of a surface sealer is a possible treatment method, although it has not been tested. A surface sealer will help prevent the penetration of moisture and limit the exposure to deicing salts.

Available Rehabilitation Methods for Alkali–Carbonate Reactivity

Rehabilitation methods to address ACR must also take into account the full extent of the deterioration. One possible method is diamond grinding, which will remove map cracking and scaling at the pavement surface. However, it will also expose another layer of concrete to moisture and deicing chemicals. Another possibility is an overlay, which will prevent direct exposure to harmful constituents while restoring ride quality. As always, rubblization or reconstruction are available methods for pavements that exhibit high-severity deterioration over a widespread area.

Selection Guidelines for Alkali–Carbonate Reactivity

As mentioned, the treatment and rehabilitation methods to address ACR are limited. Some possible methods are provided in table III-5. The selection is based solely on the severity and not the extent of the distress because ACR typically occurs over the entire pavement area. ACR initially appears as map cracking, sharing many physical manifestations with ASR. Over time, the cracking becomes much more severe and expansion-related distress becomes evident. A notable difference between ACR and ASR is that the presence of a gel reaction product is not a feature with ACR. Using the data recorded in figures I-8 through I-11 and the photo log, 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.
- Medium severity is characterized by a readily visible network of well-defined, tight map cracks. Some minor evidence of expansion may be observed.
- High severity is characterized by a well-developed pattern of open cracks. Spalling and scaling, as well as patching may be evident. Evidence of expansion-related distress should be observed.

For low-severity ACR, the only feasible treatment method is the application of a surface sealer. A surface sealer will help reduce the penetration of moisture, which contributes to swelling in the concrete. In addition, a surface sealer will limit the exposure to deicing salts, which can further exacerbate the deterioration associated with ACR. However, the effectiveness of surface sealers on pavements affected with ACR has not been studied. Therefore, agencies should experiment with the use of surface sealers before application on a wide-scale project.

For moderate-severity ACR, diamond grinding is a viable alternative for addressing map cracking caused by ACR if the distress is limited to the pavement surface. Representative cores should be taken to ensure that the distress has not progressed beyond the surface. An overlay is another feasible alternative. This option will improve the serviceability as well as inhibit further deterioration by reducing the exposure to moisture and deicing salts. If the pavement is badly deteriorated, rubblization with an overlay or reconstruction are the best options.

Sulfate Attack

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.

Table III-5. Selection of feasible alternatives to address ACR.

Severity*	Feasible Alternatives	Comments
Low	Seal the pavement	Sealing will reduce exposure to moisture and salt and thus limit further deterioration.
Moderate	Diamond grinding Overlay	These methods will temporary correct the problem but will not prevent further development.
High	Overlay Rubblization Reconstruction	Deterioration is too severe for treatment or restoration.

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

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 due to 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 III-6 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. Using the data recorded in figures I-8 through I-11 and the photo log, the severity of the distress can be assessed using the following guidance:

Table III-6. Selection of feasible alternatives to address sulfate attack.

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.

Corrosion of Embedded Steel

Corrosion of embedded steel occurs when chloride ions penetrate into the concrete, attacking the passive oxide film that helps protect reinforcing steel. The chloride ions can come from a calcium chloride accelerator, deicing salts, or seawater. The best approach to addressing corrosion is to either provide a protective coating on the reinforcing steel or use reinforcement that is not susceptible to corrosion (e.g., stainless steel, fiber reinforced polymers, etc.). This is not an option in an existing structure so most treatment methods focus on controlling the availability of moisture, oxygen, or chloride ions.

Available Treatment Methods for Corrosion of Embedded Steel

Corrosion of embedded steel can be limited by controlling the availability of moisture, oxygen, or chloride ions. The available treatment methods focus on ways to limit the amount of moisture and chloride ions into the pavement, or at least to reduce the exposure time to these harmful components. The available methods include sealing the pavement with a surface sealer and sealing joints and cracks. These methods have shortcomings that must be recognized. None will completely eliminate the exposure to moisture and chloride ions; sealing joints and cracks may be more effective at preventing salt from entering the pavement. Nonetheless, water containing chloride ions from the dissolved salt deposits can still infiltrate into the pavement.

Available Rehabilitation Methods for Corrosion of Embedded Steel

Rehabilitation methods that are available to address corrosion of embedded steel are full-depth repairs, overlays, recycling, and reconstruction. Joints that have locked-up and spalled due to corrosion of dowel bars are ideal candidates for full-depth repairs. Full-depth repairs not only remove the damaged areas but also restore joint load transfer. Corrosion of reinforcing steel at wide cracks and punchouts are also good candidates for repair.

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

Selection Guidelines for Corrosion of Embedded Steel

General guidelines, including a list of feasible alternatives to address corrosion of embedded steel, are presented in table III-7. Corrosion of embedded steel is initially manifest through cracking and staining on the pavement surface, followed by spalling. Using the data recorded in figures I-8 through I-11 and the photo log, the severity of the distress can be assessed using the following guidance:

Table III-7. Selection of feasible alternatives to address corrosion of embedded steel.

Severity	Extent	Feasible Alternatives	Comments
Low	Isolated areas	Seal the pavement Seal joints and cracks	Limit the infiltration and availability of moisture and chloride ions.
	Entire project	Seal the pavement	
Moderate	Isolated areas	Seal joints and cracks Partial depth patches[1] Full-depth patches	
	Entire project	Overlay	
High	Isolated areas	Partial depth patches[1] Full-depth repairs	In most cases deterioration is too severe for treatment; deteriorated areas must be removed and replaced.
	Entire project	Overlay Recycling Reconstruction	Deterioration is too severe and too extensive for treatment or restoration.

[1] Partial depth patching is only appropriate if the embedded steel is not a load transfer device and is located in the top third of the slab. It must be ensured that the corrosion product be completely removed from the embedded steel during the patching process.

- Low severity is characterized by fine hairline cracking over embedded steel. No scaling nor patching is observed in the affected area. Rust colored staining may be observed.
- Medium severity is characterized by open cracking over embedded steel, which may be accompanied by rust colored staining.
- High severity is characterized by spalling of the concrete over the embedded steel, which is commonly visible at the bottom of the spall. Rust colored staining will be readily visible. Corrosion of embedded steel load transfer devices can result in joint lock-up with associated cracking.

The available treatment methods for low-severity distress are limited in terms of their effectiveness. Sealing the pavement with a water-based or solvent-based silane sealer or sealing joints and cracks can provide some benefit. A similar approach is applicable to moderate-severity distress, although some patching may be required.

As the deterioration progresses to high severity, including joint spalling and possibly blowups, treatment methods become even less beneficial. Full-depth repairs are recommended for repairing spalling and blowups at transverse joints and cracks. The areas requiring repair should be isolated; repairing every joint is not cost effective. AC overlays also have their shortcomings. An AC overlay will not prevent blowups and badly spalled joints and cracks will quickly reflect through the overlay.

Partial-depth patching may offer a feasible alternative for isolated moderate- to high-severity distress over embedded reinforcing steel located in the top third of the slab depth at mid-panel locations.

High-severity spalls and blowups can also be repaired using full-depth concrete patches. However, the number of repairs must be limited if they are to be cost effective. Otherwise, reconstruction is the only viable alternative.

Selection of Preferred Alternative

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

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

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

Overall Pavement Condition

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

Just because a method is the most effective treatment or rehabilitation option for a particular MRD does not mean it is the most appropriate alternative for the pavement. For instance, the most effective method for a pavement exhibiting freeze-thaw deterioration of aggregate that is confined to a joint might be treatment with HMWM. However, if the joint is also faulted or locked-up, full-depth repair will likely be a more cost-effective option because it addresses both problems. Likewise, the selected alternative must conform to future plans of the roadway. It would be unwise to apply a treatment to address ASR when the pavement is expected to receive major rehabilitation in a few years.

Possible Constraints

Certain constraints may limit the feasibility of one or more techniques and should be considered in the selection process. At times, these factors take precedence over all other considerations, including the effectiveness of available methods. Such overriding factors may be the result of traffic, climate, materials, or construction considerations. Examples of possible constraints include the following:

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

Predicted Performance

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

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

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

Life Cycle Cost Analysis

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

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

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

Construction and Maintenance Considerations

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

Chemical Treatments

The techniques for chemical treatments vary considerably from one product to another. Special mixing, curing, and application techniques are required for each product. The manufacturer's guidelines should be closely followed. Each product should also include guidance regarding the time required before opening to traffic, the need for reapplication, and maintaining adequate surface friction. Safety precautions must be observed with some chemical treatments. Gloves, goggles, and ventilation masks are necessities for the application of some treatments. Products that can be potentially harmful should contain special safety considerations; the manufacturer should be contacted if safety guidelines are not clearly identified.

With chemical treatments, an important consideration is achieving penetration of the chemical beneath the surface. Achieving penetration through the full depth of the concrete is impossible, but careful application techniques can help increase the effectiveness. Thorough application of the chemical around and within joints and cracks, where MRD is often worse, can be beneficial. Repeated application of the chemical treatment can also help increase its long-term effectiveness.

Joint and Crack Sealing

The effectiveness of joint and crack sealing lies in its ability to prevent the occurrence of a distress or at least slow the rate of deterioration. Once MRD has progressed to moderate severity, the effectiveness of joint and crack sealing will be negligible. Where the damage is severe around joints and cracks, the process may be more harmful than beneficial. The process of refacing and cleaning joints can result in spalling in the weakened areas and therefore requires special care. Otherwise, the construction process is the same.

Crack Filling

Crack filling is not a commonly performed technique to address distress other than MRD. Preparation of the pavement is of particular importance when applying HMWM. First, all bituminous patches should be removed because HMWM deteriorates the asphalt. As with any treatment that is applied to the surface, penetration of the treatment is an important consideration. To help achieve penetration into cracks, the cracks should be thoroughly cleaned by airblasting.

Once the pavement is prepared, the treatment can be applied with a brush, spray, or squeegee. HMWM only needs to be applied in areas experiencing cracks, including surficial cracks. Within 20 minutes after application, the treatment should be covered with sand to ensure good skid resistance. Air and pavement temperatures near 20 °C are recommended to minimize curing time; curing time is typically between 3 and 6 hours. More detailed guidelines for the application of crack fillers are available (Engstrom 1994).

Surface Sealing

The application of surface sealers is another treatment that is not commonly used to address distress other than MRD. As with chemical treatments, the composition of surface sealers is continually changing as further advancements are made (Campbell-Allen and Roper 1991). The manufacturer should be consulted to ensure that the appropriate products are used and that proper application techniques are followed.

The pavement needs to be completely dry before the sealer is applied. The purpose of sealer is to keep water out of the pavement, although it can also trap water in the pavement and accelerate the deterioration. A wet surface will also prevent good penetration of the surface sealer. A clean, rough surface is preferred to allow partial penetration of the sealer. Airblasting or sandblasting may be used for this purpose. Diamond grinding is a good technique to provide these surface properties, as well as to remove surface irregularities.

Brushes, rollers, and sprayers have been used to apply surface sealers. Hand brushes have been used but are tedious and time consuming. Traffic should not be allowed on the pavement until the sealer has fully penetrated the pavement and evaporated. For silane sealers, 20 to 45 minutes is typically required (Engstrom 1994).

Partial-Depth Repairs

For partial-depth repairs, the deterioration must be limited to the upper one-third of the concrete pavement. This is not often the case with MRDs, which are typically worse at the bottom of the slab. Coring is highly recommended at representative locations to determine the extent of deterioration and to evaluate whether partial-depth repairs can be used. Before placing the patch material, the patch should be "sounded" with a hammer or rod to ensure that all of the deterioration has been removed. If the deterioration extends below the upper third of the slab, a partial-depth patch should not be placed. Rather, the entire deteriorated area should be removed, and a full-depth patch should be placed.

A wide variety of materials are available for use in partial-depth patches. These include many rapid-setting and high-early strength materials designed to reduce closure times. Material selection depends on available curing time, ambient temperature, cost, and size of the repairs. Bituminous patches are not recommended for repair of MRD on concrete pavements.

Full-Depth Repairs

Full-depth repairs are generally a better option for addressing MRD. Such distresses are typically worse at the bottom of the slab, which is exposed to moisture and deleterious chemicals for prolonged periods. Deterioration at the bottom of the slab can extend as much as 1 m beyond any visible signs of surface deterioration. Coring is recommended at representative joints and cracks to determine the extent of the deterioration and the size of patch that is required. Full-depth patches should be at least 1.8 m wide and extend the width of the traffic lane.

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

The concern with full-depth patches is that they create two joints where there was previously only one joint, thus doubling the number of locations where MRD may appear in the future. One idea is to tie both ends of the patch and place dowels at the center of the patch. After placing the patch, the patch can then be sawed at the middle. The belief is that the tied joints will not allow as much moisture and deleterious materials into the joint because it remains tight. The sawed joint at the center of the patch is surrounded by new, nonsusceptible material. Although this method can reduce the exposure at tied joints, it will not completely eliminate the recurrence of MRD, as evidenced from MRD along tied longitudinal joints. Further it is a more expensive repair and structural performance under high traffic loading may suffer at the tied joints as discussed in the previous paragraph. Another idea is to treat each of the sides of the patch with a surface sealer to help reduce the exposure to moisture.

Slab Replacement

Slab replacement involves the complete removal and replacement of a slab. This technique becomes cost effective for small areas that would require more than one full-depth repair. There are no special construction considerations for repair of MRD than for other distresses. However, if MRD is exhibited to the point where slab replacement is required, it is likely that MRD has progressed to the point where repair of single slabs would not be effective. That is, similar problems are likely to occur on the remaining slabs, so reconstruction may be a more cost-effective option.

Diamond Grinding

Diamond grinding can be performed to correct surface irregularities and to provide a smooth riding surface. Diamond grinding is also an effective technique for restoring the serviceability of the pavement in conjunction with other repair techniques. For example, the placement of full-depth repairs results in multiple repair areas, where minor spalling can occur for saw cuts and elevation differences can result from concrete placement. In such cases, diamond grinding is an effective technique for removing these surface irregularities. Diamond grinding should be performed after appropriate rehabilitation techniques (e.g., partial-depth or full-depth repairs) but before any surface treatment methods.

Overlays

The most important construction consideration for overlays is the amount of pre-overlay repair work that needs to be conducted. The performance of the overlay is largely dependent on the type and amount of pre-overlay repair. The placement of an AC overlay requires that all badly deteriorated areas (generally moderate- and high-severity distresses) be repaired. The problem with MRD is that it often affects the entire pavement area, in which case the repairs can become too costly. In such cases, an AC overlay can be used to improve the serviceability until more extensive rehabilitation can be conducted. In addition, the MRD is likely to continue to progress, which will further contribute to the deterioration of the overlay.

The placement of an unbonded PCC overlay requires less pre-overlay repair and is therefore a better option on pavements exhibiting extensive distress. Fracturing of the existing pavement is also an option to further reduce the interaction with the overlay. Although the amount of pre-overlay repairs is reduced, unbonded PCC overlays require thicker sections than other overlay types, which offsets the cost. In addition, the thicker sections may create problems with grade elevations and bridge clearances, which can be extremely costly.

Pavement Reconstruction

In terms of construction techniques, reconstruction of pavements exhibiting MRD does not require any special considerations over pavements exhibiting other distresses. However, special considerations for PCC mix designs are required to prevent the durability problem from recurring. If possible, an alternate aggregate source should be used when the distress is due to a materials problem. If an alternate aggregate source is not available, then the mix design should be adjusted to account for the potential problem as discussed in the next chapter of this guideline.

Pavement Recycling

Recycling of the pavement offers an alternative way to conduct reconstruction. This alternative conserves aggregate resources and can potentially result in substantial cost savings, especially in areas where aggregate resources are scarce. Special mix design considerations are required for any recycling project, but additional considerations are required if the existing pavement also exhibits a materials or durability problem. These considerations are addressed in the next chapter.

Summary of Treatment and Rehabilitation Methods

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

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

MATERIALS AND MIX DESIGN FOR PREVENTION OF MRD IN CONCRETE PAVEMENTS

This section of the guideline considers specific mix design and construction factors that have a direct influence on the production of durable concrete pavements. It is not designed to replace existing mix design and construction practices, such as those advocated by the Portland Cement Association (Kosmatka et al. 2002) and the American Concrete Institute (ACI) (1991), 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.

This section of the guideline describes how considering the selection and proportioning of constituent materials and applying of proper construction techniques can significantly reduce the incidence of MRD in concrete pavements. It also provides specific considerations related to addressing individual MRDs. It is noted that this section is not written to replace the vast body of knowledge that already exists, but instead to supplement it by presenting relevant information in a usable format. The user of this guideline must rely on information contained in 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]. Local practices, specifications, and experience are also important and should be consulted.

Constituent Materials Selection for Preventing MRD

The selection of materials for durable concrete is based on the premise that quality concrete can only be produced from quality materials. In the most basic conceptualization, hardened concrete is composed of aggregates and paste. Aggregates are usually classified as being either fine or coarse as defined in ASTM C 33. The paste is composed of hydrated portland cement, water, and air, and may also contain additions and admixtures (or their remnants). Common admixtures are added to entrain air or to modify the properties of the fresh concrete (e.g., accelerate set, delay set, or modify the rheology). Additional cementitious or pozzolanic materials such as fly ash, silica fume, or ground granulated blast furnace slag (GGBFS) can also be added. Each of these components must be carefully selected to produce a mixture that is readily mixed, placed, and consolidated without excessive bleeding. Upon hardening, this mixture must be dense, relatively impermeable, and resistant to environmental effects and deleterious chemical reactions over the length of its service life.

In the following sections, each concrete constituent material is discussed in the context of overall concrete durability. The user of this guideline is directed to the referenced documents for a more in-depth discussion of the subject matter.

Aggregate Selection

Aggregates typically make up 60 to 75 percent of the total volume (70 to 85 percent by weight) of concrete (Kosmatka et al. 2002). Thus, the properties of the aggregate will have a profound effect on the durability of the concrete pavement. Although aggregates are commonly considered inert filler within the concrete structure, this is rarely the case. Aggregates in concrete must be able to resist the forces exerted on them by the environment without incurring damage to themselves or the surrounding paste. This may include physical mechanisms such as freezing and thawing and/or moisture cycling. There are also deleterious chemical reactions in which aggregates play an active role, the most common being alkali–aggregate reactions, but aggregates can also contribute to internal sulfate attack or other chemical distress mechanisms. As a result, care must be exercised in selecting aggregates that not only possess adequate strength, but are also physically and chemically stable within the concrete and its environment. This section of the guideline addresses the physical and chemical characteristics of the aggregate.

The aggregate selected for use in paving concrete must meet the requirements of ASTM C 33, although this alone does not ensure concrete durability (ACI 1992). One of the most important factors in selecting durable concrete aggregate is demonstrated field performance under similar conditions. Unfortunately, this approach has some pitfalls, including the following:

- Aggregate materials often possess varying properties, even within the same pit or quarry. This is particularly true of gravels that are quite variable, but also is true of quarried sources.
- Cement properties have varied over time. For example, a recent study found that the alkalinity of cement has increased slightly since the 1950s, a factor that may contribute to greater potential for alkali–aggregate reactivity for marginal aggregates (PCA 1996). More importantly, the composition of a given type of cement can vary considerably while still meeting the requirements of ASTM C 150.
- Pavement construction practices have changed. Shorter mixing times, more rapid placement, stiffer consistency, and changes in curing practices may all impact long-term durability of concrete pavements. Within a given location, some of these changes may be sufficient to cause MRD when marginal aggregates are used.
- Concrete mixture design changes, particularly the use of smaller maximum aggregate, increased cement contents (thus more paste), and chemical and mineral admixtures can contribute to deleterious reactions with aggregate that previously did not occur.

Also, some sources of durable, high quality aggregates have become completely exhausted and are therefore unavailable. The need to use aggregate from new sources by definition means no previous field performance experience exists. Thus laboratory testing of the aggregate for durability is required.

Table III-8 provides a list of desirable characteristics and standard tests that can be conducted to evaluate aggregate (Kosmatka et al. 2002). A more recent list of tests is provided in a *Guide for Use of Normal Weight and Heavyweight Aggregates in Concrete* (ACI 1996). Some of these tests are run solely on the aggregate particles, such as ASTM C 295, *Practice for Petrographic Examination of Aggregates in Concrete*, whereas others examine the performance of concrete containing the aggregates under evaluation, such as ASTM C 666 Method A, *Test Method for Resistance of Concrete to Rapid Freezing and Thawing*. Because of the limitations of all available test methods, it is good practice to avoid the use of aggregate sources with demonstrated poor field performance even if laboratory testing is satisfactory (PCA 1995) unless other reasons for the poor performance can be documented. Also, some additional standard tests have been added since the publication of this document.

Figure III-2 presents a flowchart that can be used to evaluate aggregate sources. It is assumed that the aggregate source under study has already passed ASTM C 33 requirements. Many agencies have developed their own approaches for evaluating aggregates, commonly focusing on a single distress mechanism. For example, numerous procedures that recently been developed to evaluate the ASR susceptibility of an aggregate source including the proposed American Association of State Highway and Transportation Officials (AASHTO) Guide Specification (AASHTO 2000) and the new Canadian Standards (Fournier et al 1999). Each agency should evaluate their procedures and modify them as necessary to include methods that best address local conditions.

The following text briefly discusses tests presented in figure III-2. It is assumed that the aggregate source under study has already passed ASTM C 33 requirements. The list of standards and test methods presented is by no means exhaustive, as additional tests have also been found to be useful by various transportation agencies and research institutions.

ASTM C 295: Standard Guide for Petrographic Examination of Aggregates for Concrete

This standard should be considered as routine practice for examination of aggregates being considered for use in concrete pavements. It requires the use of optical microscopy (OM) and may also employ additional procedures such as x-ray diffraction (XRD) analysis, differential thermal analysis, and infrared spectroscopy, among others. This standard also requires the services of a qualified petrographer. Two excellent references that will assist in conducting this test procedure are *Petrographic Evaluation of Concrete Aggregates* by Mielenz (1994) and *Handbook of Concrete Aggregates* by Dolar-Mantuani (1982).

This standard will identify the constituents of an aggregate sample, which in some cases can be linked to the expected behavior of these aggregates in the field. Both physical and chemical properties of the material can be identified and classified, and the relative amounts of the constituents can be determined (this is particularly important for gravel deposits). This method is very useful in comparing aggregate from new sources to that of existing sources that have test and field performance data available. In considering the durability of concrete, a petrographic examination of the aggregate will provide valuable information on the potential freeze-thaw

Table III-8. Aggregate characteristics and test methods (Kosmatka et al. 2002).

Characteristic	Significance	Test designation*	Requirement or item reported
Resistance to abrasion and degradation	Index of aggregate quality; wear resistance of floors and pavements	ASTM C 131 (AASHTO T 96) ASTM C 535 ASTM C 779	Maximum percentage of weight loss. Depth of wear and time
Resistance to freezing and thawing	Surface scaling, roughness, loss of section, and aesthetics	ASTM C 666 (AASHTO T 161) ASTM C 682 AASHTO T 103	Maximum number of cycles or period of frost immunity; durability factor
Resistance to disintegration by sulfates	Soundness against weathering action	ASTM C 88 (AASHTO T 104)	Weight loss, particles exhibiting distress
Particle shape and surface texture	Workability of fresh concrete	ASTM C 295 ASTM D 3398	Maximum percentage of flat and elongated particles
Grading	Workability of fresh concrete; economy	ASTM C 117 (AASHTO T 11) ASTM C 136 (AASHTO T 27)	Minimum and maximum percentage passing standard sieves
Fine aggregate degradation	Index of aggregate quality; Resistance to degradation during mixing	ASTM C 1137	Change in grading
Uncompacted void content of fine aggregate	Workability of fresh concrete	ASTM C 1252 (AASHTO T 304)	Uncompacted voids and specific gravity values
Bulk density (unit weight)	Mix design calculations; classification	ASTM C 29 (AASHTO T 19)	Compact weight and loose weight
Relative density (specific gravity)	Mix design calculations	ASTM C 127 (AASHTO T 85) fine aggregate ASTM C 128 (AASHTO T 84) coarse aggregate	—
Absorption and surface moisture	Control of concrete quality (water-cement ratio)	ASTM C 70 ASTM C 127 (AASHTO T 85) ASTM C 128 (AASHTO T 84) ASTM C 566 (AASHTO T 255)	—
Compressive and flexural strength	Acceptability of fine aggregate failing other tests	ASTM C 39 (AASHTO T 22) ASTM C 78 (AASHTO T 97)	Strength to exceed 95% of strength achieved with purified sand
Definitions of constituents	Clear understanding and communication	ASTM C 125 ASTM C 294	—
Aggregate constituents	Determine amount of deleterious and organic materials	ASTM C 40 (AASHTO T 21) ASTM C 87 (AASHTO T 71) ASTM C 117 (AASHTO T 11) ASTM C 123 (AASHTO T 113) ASTM C 142 (AASHTO T 112) ASTM C 295	Maximum percentage allowed of individual constituents
Resistance to alkali reactivity and volume change	Soundness against volume change	ASTM C 227 ASTM C 289 ASTM C 295 ASTM C 342 ASTM C 586 ASTM C 1260 (AASHTO T 303) ASTM C 1293	Maximum length change, constituents and amount of silica, and alkalinity

* The majority of the tests and characteristics listed are referenced in ASTM C 33 (AASHTO M 6/M 80). ACI 221R-96 presents additional test methods and properties of concrete influenced by aggregate characteristics.

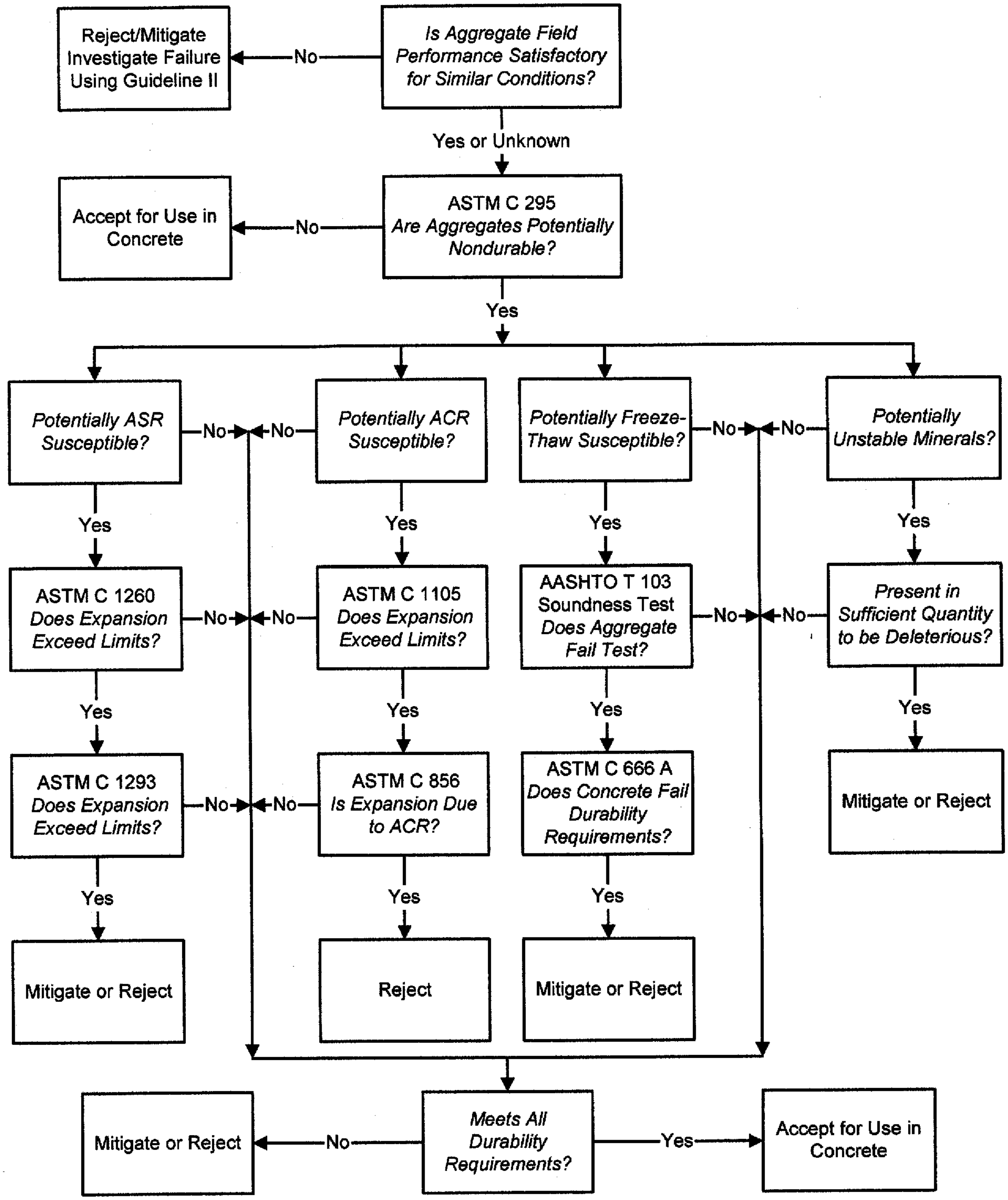


Figure III-2. Flowchart for the selection of durable aggregates.

durability of the aggregate, whether chemically unstable minerals are present, and whether alkali reactive minerals are present.

For example, it is known that finely porous and highly weathered, or otherwise altered coarse aggregate particles can be especially susceptible to freeze-thaw damage. This damage is either manifested through fracturing of the particle or the surrounding paste resulting in what is commonly called D-cracking of the pavement. Finely porous aggregate near the pavement surface can also suffer freeze-thaw damage in the form of popouts. For pavements being constructed in regions experiencing freezing and thawing cycles, petrography can be used to identify potentially susceptible materials within the coarse aggregate sources. Thus, this standard can provide an initial screening in the aggregate selection process. Furthermore, the petrographic properties of aggregates with known performance records can be compared with new sources to make an initial assessment of the new aggregate's suitability for use in concrete pavement construction.

It is recommended by some that aggregates with high absorption (greater than 2 percent in 24 hours) should not be used in a freeze-thaw environment (Pigeon and Plateau 1995). This is because even though the aggregates may maintain their integrity under freezing and thawing, they may rapidly expel water that can either fracture the paste or dissolve soluble paste components. Therefore, aggregates having questionable petrographic or absorption characteristics should be tested in a confined state (i.e., embedded in concrete) using test methods such as ASTM C 666 Method A or C 682 (which are described later).

Chemically unstable minerals, such as soluble sulfates and unstable sulfides, or volumetrically unstable materials such as smectites can be readily identified using petrographic means. If present in sufficient quantity, these minerals can have a deleterious effect on hardened concrete. Soluble sulfates can lead to internal sulfate attack, whereas unstable sulfides can form sulfuric acid, resulting in acid attack. Volumetrically unstable materials will shrink and swell under moisture cycles resulting in particle and/or paste degradation.

In addition, identification of many alkali–silica reactive constituents can be accomplished using this standard. Commonly recognized alkali–silica reactive constituents include opal, cristobalite, tridymite, siliceous and some intermediate volcanic glass, chert, glassy to cryptocrystalline acid volcanic rocks, synthetic siliceous glass, some argillites, phyllites, metamorphic graywackes, schists, gneisses, gneissic granites, vein quartz, quartzite, and sandstone. It is noted that this list does not identify all constituents of North American aggregates that are potentially reactive (PCA 1995). An excellent summary of the petrography of alkali–silica reactive aggregates is provided in ACI 221.1R-98 (ACI 1998). If more than the following quantities of constituents are observed in the fine and coarse aggregate, it shall be considered potentially reactive (PCA 1995):

- Optically strained, microfractured, or microcrystalline quartz exceeding 5.0 percent (a common constituent of granite and granite gneiss).
- Chert or chalcedony exceeding 3.0 percent.
- Tridymite or cristobalite exceeding 1.0 percent.
- Opal exceeding 0.5 percent.
- Natural volcanic glass in volcanic rocks exceeding 3.0 percent.

If ASTM C 295 indicates that the aggregate is potentially reactive according to the above criteria, or if potential for alkali–silica reactivity is suspected, ASTM C 1260 and/or C 1293 should be conducted. It is noted that a petrographic examination may not detect small amounts of reactive material and that the results should therefore be confirmed by expansion tests as will be discussed later (ACI 1998).

Petrographic evaluation of aggregate can also be used to identify potential alkali–carbonate reactive aggregate. These are usually calcareous dolomites or dolomitic limestones with clayey insoluble residues. Some alkali–carbonate reactions can occur in dolomites and very fine-grained limestones that are essentially free of clay, but it is uncertain whether these reactions are deleterious. The structure and texture of reactive carbonate aggregates is rather unique and identifiable (Dolar-Mantuani 1982). Identification of dolomite in a fine-grained carbonate rock makes it desirable to run ASTM C 586 and/or ASTM C 1105 (ACI 1998).

ASTM C 586: Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)

This test method is used to determine the expansive characteristics of carbonate rocks immersed in a NaOH solution. This test should be conducted if the potential for ACR is suspected or if ASTM C 295 indicates that potentially reactive constituents are present. The test is fairly rapid and is an effective tool for screening aggregate sources (ACI 221.1R-98). Small cylinders (35 mm long by 9 mm diameter) of the suspect rock are immersed in a 1 N NaOH solution at room temperature. The change in length of the specimen is measured at 7, 14, 21, and 28 days of immersion, and at 4-week intervals thereafter. If the test is continued beyond 1 year, measurements should then be made at 12-week intervals. Generally, expansive tendencies are observed during the first month (Farny and Kosmatka 1997). A 28-day expansion equal to or exceeding 0.10 percent indicates the potential for deleterious expansion in the field. Results of this test can be used qualitatively to predict expansion of concrete in the field, but quantitative predictions of concrete expansion are not possible. This test should therefore not be used alone, but instead in conjunction with other tests such as ASTM C 295 and C 1105 to predict whether an aggregate will likely be susceptible to ACR.

ASTM C 666: Test Method for Resistance of Concrete to Freezing and Thawing

This test method is used to assess the resistance of concrete specimens to rapidly repeated cycles of freezing and thawing. It is considered by many to be the best available test method for evaluating freeze-thaw resistance of aggregate, but it is not without its critics (Pigeon and Plateau 1995; ACI 1992). In this test method, concrete beams are prepared with the aggregate under evaluation and subjected to rapid freezing and thawing cycles. In Procedure A, the specimens are frozen and thawed in water whereas in Procedure B freezing occurs in air while thawing is done in water. Procedure A is the preferred method. Many SHAs have modified this procedure to address their specific needs and observations.

Deterioration is measured through the reduction in the dynamic modulus of elasticity of the concrete, the linear expansion of the specimen, and/or through the weight loss incurred. According to the PCA (1992), a number of SHAs use an expansion failure criterion of 0.035 percent in 350 freeze-thaw cycles or less to help indicate whether or not an aggregate is susceptible to freeze-thaw deterioration.

Criticism of the test method primarily centers on the fact that it is not representative of actual field conditions. The concrete is saturated and then subjected to rapid freezing and thawing, which is unlikely to occur in the field. Thus, although the test is able to rank aggregate from excellent to poor, it cannot be used reliably to predict the field performance of marginal aggregate (ACI 1992). Because the test is more severe than actual field conditions, aggregates that pass this test are generally going to perform well in the field. But it may reject aggregate that has established good field performance.

As noted in the TRB Circular entitled *Durability of Concrete*, only Procedure A in the standard should be used; Procedure B should never be used (TRB 1999).

ASTM C 1105: Test Method for Length Change of Concrete Due to Alkali–Carbonate Rock Reaction

If ASTM C 295 and C 586 indicate that an aggregate is potentially alkali–carbonate reactive, ASTM C 1105 should be conducted. It is recognized as the best indicator of potentially deleterious expansion due to carbonate aggregate in concrete (ACI 1998). This test calls for six concrete specimens to be fabricated using the aggregate under evaluation and the job cement, if possible. The specimens are kept in moist storage and their change in length is measured at 7, 28, and 56 days, and at 3, 6, 9, and 12 months. In the appendix of the ASTM standard, average expansions equal to or greater than 0.015 percent at 3 months, 0.025 percent at 6 months, and 0.030 percent at 1 year are considered indicative of a potentially deleteriously aggregate source. The appendix in the standard suggests that if the aggregate is judged potentially reactive, the following measures can be employed to control the effects of the reaction:

- Use the smallest nominal maximum coarse aggregate practical.
- Use a maximum of 20 percent reactive rock in the coarse aggregate, a maximum of 20 percent reactive rock in the fine aggregate, or a total of 15 percent reactive rock in the coarse and fine aggregate combined.
- Use cement having an alkali content below that of cement for which the results indicate that the aggregate is nonreactive. However, some have noted that this is not very effective in controlling ACR (Farny and Kosmatka 1997).

Prevention of ACR is discussed in detail later in this chapter.

ASTM C 1260: Test Method for Potential Alkali Reactivity of Aggregates (Mortar Bar Method)

This test method is specifically designed to establish the aggregates' potential for alkali–silica reactivity. In this test, the aggregate is sized and cast into mortar bars at a water to cementitious material ratio (*w/cm*) of 0.47. Specimens undergo 2 days of moist room and water cure, and are then submerged in a 1 N NaOH solution for 14 days. Expansion is measured at 1, 3, 7, 10, and 14 days. The total duration of the test is 16 days.

The following 14-day expansion criteria are presented in the appendix of C 1260 to judge potential reactivity:

- Expansion greater than 0.20 percent is indicative of potentially deleterious expansive reactions.
- Expansions between 0.10 and 0.20 percent have been observed for aggregates with variable field performance. Supplemental information must be developed.
- Expansions less than 0.10 percent are indicative of innocuous behavior in most cases.

Some granitic gneisses and metabasalts have been found to be deleterious in the field even though expansion in ASTM C 1260 has been less than 0.10 percent. Thus, unless good field performance can be demonstrated for these types of aggregate, the aggregate in question should be considered potentially reactive.

Because the specimens are submerged in 1 N NaOH solution, the alkali content of the cement is relatively unimportant. Also, the test will reveal potential reactivity, but will not predict field performance because mixture alkalinity is not considered. But the rapidity of this test and its ability to detect reactivity in slowly reactive aggregates makes it very attractive. In combination with ASTM C 295 and evaluation of field performance records, most alkali-silica reactivity problems can be anticipated and mitigated (Stark 1994). If the expansion falls between 0.10 and 0.20, the aggregate should be tested using ASTM C 1293.

It is noted in the standard that specimens undergoing apparently deleterious expansion should undergo further analysis to determine if the expansion is indeed a result of ASR. ASTM C 856, *Practice for Petrographic Examination of Hardened Concrete*, is one recommended procedure to identify the alkali-silica gel in the test specimen.

In closing, ASTM C 1260 is considered to be a quick and reliable test for characterizing the potential for reactivity of slowly as well as rapidly reactive rock types (ACI 221.1R-98). But it is a severe test that may reject acceptable aggregates, possibly necessitating long-term testing using ASTM C 1293.

ASTM C 1293: Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali–Silica Reaction

In this test procedure, concrete prisms made of the aggregate under evaluation are constructed with a Type I high alkali cement plus sufficient reagent quality NaOH to simulate a 1.25 percent Na_2O equivalent cement. The concrete proportions are strictly defined with a cement content of 420 ± 10 kg/m^3 and a water-to-cement ratio (*w/c*) of 0.42 to 0.45 being specified. The specimens are sealed in containers and are suspended above water at 38°C. Length measurements are made at 7, 28, and 56 days and at 3, 6, 9, and 12 months and every 6 months thereafter for as long as desired. These are compared to the 1-day length and used to determine if expansion due to ASR is occurring. Aggregates in specimens having expansion in excess of 0.04 percent after 1 year are considered potentially reactive.

As with ASTM C 1260, it is recommended that if deleterious expansion is observed, ASTM C 856 should be used to confirm that the cause of expansion is ASR. ASTM C 1293 can be used to assess both fine and coarse aggregate. Due to its long duration, it is also more effective than ASTM C 1260 for assessing slowly reactive aggregates.

AASHTO T 103: Standard Method of Test for Soundness of Aggregates by Freezing and Thawing

This test method is conducted on unconfined coarse aggregate to evaluate freeze-thaw susceptibility. It is used to initially screen aggregate. Aggregates that fail this test are not permitted for use in concrete. Those that pass this test are subjected to further testing. In figure III-2, this would be ASTM C 666 (Method A).

Each SHA would need to establish failure criteria based on experience. For example, in the Kansas Department of Transportation version of this test, a coarse aggregate specimen is prepared containing material passing the 25-mm sieve and retained on the 2.36-mm sieve. The gradation is carefully selected and recorded. The specimen is then saturated and cycled between freezing and thawing for 25 cycles. After 25 cycles, the cumulative percentage of material retained on each of the four sieves (19, 9.5, 4.75, and 2.36 mm) is determined. A freeze-thaw loss ratio is calculated by dividing the sum of cumulative percentages of aggregate retained on each sieve after freeze-thaw cycling to that for the original aggregate gradation. If the calculated freeze-thaw loss ratio is less than 0.85, the aggregate is rejected.

Summary of Aggregate Selection Process

This section provided a methodology based on standards and test methods that can be used to evaluate the potential durability of aggregate for use in concrete pavements. It is assumed that the aggregates under evaluation pass the requirements of ASTM C 33. In evaluating an aggregate source for durability, it is important that ASTM C 295 first be used to characterize the physical and chemical properties of the aggregate. Supplemental testing is then conducted to further define the physical and chemical stability of aggregate. Demonstrated field performance is a crucial element of any study of aggregate durability, especially in light of the limitations of available test methods. Agencies are therefore exhorted to maintain detailed records of all aggregate investigations and attempt to correlate these investigations with field performance.

Cementitious and Pozzolanic Materials

The selection of cementitious and pozzolanic material is an extremely important element in designing durable concrete pavements. In many applications, the use of a standard ASTM C 150 Type I cement will provide satisfactory results. But the designer should not take durability for granted, and therefore must carefully consider the properties of the cement and additional cementitious/pozzolanic materials in the context of the long-term physical and chemical stability of the concrete pavement. This section describes important factors to consider when selecting cementitious/pozzolanic materials for use in a pavement project. Cements identified through ASTM C 150 and C 595M are described, as well as fly ash, slag, and silica fume additives with the emphasis on enhancing concrete durability.

Cement

The standard specification for portland cements used in the United States is presented in ASTM C 150. The following cement classifications are presented in the ASTM standard:

- Type I–For use when the special properties specified for any other type are not required.
- Type IA–Air-entraining cement for the same uses as Type I, where air-entrainment is desired.
- Type II–For general use, particularly when moderate sulfate resistance or moderate heat of hydration is desired.
- Type IIA–Air-entraining cement for the same uses as Type II, where air-entrainment is desired.
- Type III–For use when high early strength is desired.
- Type IIIA–Air-entraining cement for the same uses as Type III, where air-entrainment is desired.
- Type IV–For use when low heat of hydration is desired.
- Type V–For use when sulfate resistance is desired.

The most common cement employed in pavement construction in the United States is Type I. The required chemical properties of Type III cements are similar to Type I, but Type III cements are ground finer to promote the development of higher early strength. Type III cements are gaining more widespread use, particularly in applications where high early strength is needed such as for rapid-setting repairs and "fast track" construction. In areas where external sulfate attack is a problem, Type V cements are used. Air-entrained cement, designated with an "A," has small quantities of air-entraining material interground with the clinker during manufacture.

In addition to these cements, ASTM C 595M provides standard specifications for blended hydraulic cements. These specifications recognize the following five classes of blended cements:

- Type IS–Portland blast-furnace slag cement
- Type IP and Type P–Portland-pozzolan cement
- Type I (PM)–Pozzolan modified portland cement
- Type S–Slag cement
- Type I (SM)–Slag modified portland cement

These cements are formed by intimately blending portland cement with fine materials such as GGBFS, fly ash or other pozzolans, hydrated lime, and pre-blended cement combinations of these materials (Kosmatka et al. 2002). In the past, these have not been commonly used on paving projects in the United States, but they may see increased use to produce durable concrete pavements, particularly Type IS, Type IP, and Type I (PM). Each of these three can be further designated as air-entraining (A), moderate sulfate resistant (MS), and/or moderate heat of hydration (MH). For example, an air-entraining, moderate sulfate resistant pozzolan modified portland cement would be designated as Type I (PM)-A(MS). There are many references available that provide detailed descriptions of the physical and chemical characteristics of cements. One such reference is *Design and Control of Concrete Mixtures* by the PCA (Kosmatka et al. 2002).

It is known that variations in the physical and chemical characteristics of cement can have an impact on the initiation and progression of MRD in concrete pavements. A recently completed study by the PCA (1996) confirmed that slight changes in cement properties have occurred since the 1950's. Cement produced in the 1990's achieves strength more quickly in the first 7 days, but has reduced long-term strength gain than its 1950's counterpart. This is partially a result of the fineness of modern cements, which in all cases are more finely and uniformly ground than cements from the 1950's. But chemistry also plays a role. Modern cements have greater amounts of C_3S (3 to 10 percent more) and less C_2S (5 to 14 percent less) than cements from the 1950s. This contributes to faster early strength gain at the expense of long-term strength development. Also, sulfate content for modern cements is 0.5 to 1.2 percent higher and the total alkali content has increased by an average of 0.08 percent (Na equivalent) for Type I cement. The increase in sulfate can be partially accounted for by the increased demand for gypsum necessitated by finer grinding of the cement, but changes in processing and raw materials and the use of waste fuels are also contributing factors (Gress 1997). Although the results of the PCA report do not show dramatic changes in cement properties, some have voiced concern that the combination of small changes in cement characteristics may negatively affect the durability of concrete.

As a result, some researchers are suggesting that the movement toward quicker setting cement and faster construction has been detrimental to long-term concrete durability. Mehta (1997), for example, argues that for concrete to remain durable, it must remain relatively impermeable. He proposes that durable concrete should be slow-hardening with high creep and low elastic modulus at early ages to resist drying shrinkage and thermal cracking. Although he acknowledges that this can be accomplished through the use of coarser ground cement with low C_3S content, he doesn't directly advocate a return to the cement of the past. He instead suggests that there is an overemphasis on speed of construction that has resulted in the unnecessary use of high early-strength mixtures. To address this, instead of using Type I or Type III portland cement, blended cements containing slag and/or fly ash could be used. The lower heat of hydration and denser, less soluble microstructure will produce less permeable, more durable concrete. This would necessitate a major change in the way concrete quality is judged, shifting strength requirements to 56 days or 90 days instead of the 28 days commonly used today. The use of mixtures having slower strength gain of this type may be practical for many paving projects, although they are not practical in situations where early opening is required. It must be realized that speed of construction and long-term concrete durability both need to be considered. Thus, when selecting cementitious materials, the designer should not focus exclusively on 28-day strength gain as the measure of concrete quality. And unless the constraints of the project demand high early strength (i.e., high user costs dictate early opening), the use of blended cements or pozzolanic replacement should be considered.

Cement properties related to the development of specific MRD types are discussed later in this section of this guideline, which also provides detailed information regarding mix design and construction practices that can be exercised to control specific MRD in concrete pavements.

Fly Ash

Fly ash is an industrial byproduct produced from the burning of coal. It is primarily silica glass containing silica, alumina, iron, and calcium, with magnesium, sulfur, sodium, potassium, and carbon as minor constituents (Kosmatka et al. 2002). Fly ash is classified according to ASTM C 618 as either Class C or Class F, the primary difference being the minimum percentage of silicon dioxide, aluminum oxide, and iron oxide present (combined total of 50 and 70 percent, respectively). The burning of bituminous or anthracite coals tends to produce low calcium fly ash (containing less than 10 percent analytic CaO) that has historically been classified as Class F.

Alternatively, Class C fly ash is typically high in calcium (containing 15 to 35 percent CaO), being the product of the combustion of subbituminous or lignite coals (Mehta and Monteiro 1993; Dewey et al. 1996).

Mehta and Monteiro (1993) provide an excellent description of the differences between high and low calcium fly ash. High calcium fly ash (typically ASTM C 618 Class C) is both cementitious and pozzolanic. It consists mostly of silicate glass containing calcium, magnesium, aluminum, and alkalies, with small amounts of crystalline material present in the form of quartz and C_3A. Free lime and periclase may also be present, as may $C\overline{S}$ and $C_4A_3\overline{S}$ if high sulfur coals were used. The particle size distribution has 10 to 15 percent larger than 45μm with a Blaine fineness of 300 to 400 m^2/kg. Most particles are solid spheres having diameters less than 20 μm. The particle surfaces are generally smooth but are not as clean as that found on low calcium fly ash, possibly due to the deposition of alkali-sulfates on the surface (Mehta and Monteiro 1993).

Low calcium fly ash (typically classified as ASTM C 618 Class F) is mostly silicate glass containing aluminum, iron, and alkalis (Mehta and Monteiro 1993). Small quantities of crystalline material exist in the form of quartz, mullite, sillimanite, hematite, and magnetite. The particle size distribution is typically slightly coarser than that of the high calcium fly ashes, with 15 to 30 percent larger than 45 μm, a Blaine fineness of 200 to 300 m^2/kg, and having an average diameter of 20 μm. Most of the particles are solid, but cenospheres (completely hollow, empty spheres) and plerospheres (hollow spheres packed with numerous small spheres) may be present (Mehta and Monteiro 1993).

Fly ash can be incorporated into concrete either through blending or intergrinding with portland cement and classified under ASTM C 595M as Type IP, Type P, or Type I(PM) or introduced during mixing either as an addition to the portland cement or as a partial replacement.

It is widely recognized that concrete can benefit greatly from the inclusion of fly ash, either as an addition to or replacement of portland cement. Because of its smooth spherical shape and broader size distribution than ASTM Type I portland cement, fly ash acts as a "microaggregate," packing into spaces that would normally be left empty. This improves workability and reduces the water demand of the fresh concrete, and generally results in less bleeding and segregation and improved finishability (Kosmatka et al. 2002; Mehta and Monteiro1993).

The use of fly ash can also produce a denser, less permeable microstructure that is less susceptible to chemical attack. In addition to the physical contribution of improved particle packing, the hydration of the pozzolan will further reduce permeability by filling remaining pore space with hydration products. The pozzolanic reaction that occurs with low calcium fly ash can significantly reduce ASR by converting calcium hydroxide (CH) to calcium silicate hydrate (CSH) and chemically tying up the alkalis in the concrete (Kosmatka et al. 2002). Similarly, sulfate attack can also be reduced through the addition of low calcium fly ash. Once again, the pozzolanic reaction will combine the silica in the fly ash with CH and alkalis, reducing the potential for deleterious gypsum formation (ACI 1992).

However, the benefit offered by a specific fly ash is highly dependent on its individual characteristics. It is widely acknowledged that the current ASTM C 618 classification of coal fly ash is inadequate to describe whether the addition of a fly ash will be beneficial to the performance of concrete. Not only is specific knowledge regarding chemical and physical characteristics important, but also the fly ash's mineralogical structure must be identified to understand its reactivity (Dewey et al. 1996). Additionally, the amount of CaO present has a significant effect on the ability of the fly ash to mitigate the initiation and progression of certain MRD such as ASR (Gress 1997). And, in some instances, sulfates and alkalis present in fly ash may actually be detrimental to concrete durability.

It is therefore necessary to carefully select fly ash for a specific concrete application and demonstrate through testing that it will have a beneficial impact on performance. Many studies have found that different volumes of similarly classified fly ashes are required to gain the same desired enhancement, and that an optimal volume for one particular fly ash may actually be detrimental to concrete durability for another fly ash.

Class C fly ashes have been particularly problematic as far as concrete durability is concerned. Because of their higher CaO content, they do not react with CH and alkalis to form CSH to the same degree as lower calcium Class F fly ashes. Thus, they are widely recognized as not being as effective in controlling ASR or sulfate attack, and in some cases, may be detrimental (ACI 1992; Kosmatka et al. 2002; Farny and Kosmatka 1997). Also, Class C fly ashes contain more reactive crystalline compounds such as C_3A, $C\overline{S}$, and $C_4A_3\overline{S}$ and more alkalis and sulfates than do low calcium Class F fly ashes (Mehta and Monteiro 1993). Finally, both Class C and F fly ashes also commonly contain small percentages of unburned carbon. If present in quantities greater than 5 percent, this unburned carbon will result in increased water demand to maintain a given consistency and increased quantity of air-entraining admixture to obtain a desired air content (Mehta and Monteiro 1993).

Recommendations regarding the addition of fly ash to concrete to mitigate specific MRD in concrete pavements are reserved for later sections of this guideline. In general, it is evident that fly ash can be a powerful and cost-effective ally in producing durable concrete through physical and chemical modification to its microstructure. The addition of low calcium fly ash will reduce concrete permeability while converting soluble CH into less soluble, more stable CSH. It is important to demonstrate the ability of the fly ash to mitigate ASR and increase sulfate resistance through standard tests such as ASTM C 227 and C 1293 for ASR and ASTM C 1012 for sulfate resistance.

The current state of knowledge suggests that the use of Class C fly ash should be discouraged when attempting to mitigate potential MRD problems such as ASR or sulfate attack. Also, the fly ash's contribution of sulfates, aluminum, and alkalis to the concrete must be considered. In all cases, concrete mixtures containing fly ash should be tested to ensure that adequate durability exists. In areas subjected to deicer applications, special care must be taken to ensure that a high cementitious material content and low *w/cm* are used. Sufficient curing time to account for the slower hydration rate of concrete containing fly ash is required prior to deicer application to prevent scaling.

Other Cementitious/Pozzolanic Materials

Two other industrial byproducts that may be added to concrete as supplemental cementitious/pozzolanic materials are GGBFS and silica fume. Each may be beneficial in preventing MRD in some situations.

Ground Granulated Blast Furnace Slag

GGBFS is a byproduct of the production of cast iron, in which the molten slag (at a temperature of 1500 °C) is rapidly chilled by quenching in water. This forms a glassy, sand-like material that is then finely ground to less than 45 μm, having a surface area of 400 to 600 m^2/kg Blaine (Kosmatka et al. 2002). GGBFS is nonmetallic, consisting mostly of silicate glass containing calcium, magnesium, aluminum, and silicate with potentially small quantities of crystalline compounds of melilite present (Mehta and Monteiro 1993). This rough textured material is cementitious in nature, meaning that it possesses hydraulic cementing properties. When combined with portland cement, the NaOH or CaOH activates the GGBFS, which hydrates and sets in a manner similar to portland cement (Kosmatka et al. 2002). Specifications for GGBFS for use in concrete are provided in ASTM C 989.

GGBFS may be blended or interground with the cement and classified according to ASTM C 595M as Type IS, Type S, or Type I(SM). Alternatively, it may be added during batching as supplemental cementing materials or as a partial replacement for portland cement.

In producing durable concrete, the addition of GGBFS has had positive effects. Like fly ash, it reduces the permeability of concrete and therefore should have a beneficial effect on durability by limiting the penetration of harmful agents, decreasing the solubility of the paste, and preventing the rapid movement of pore solution within the concrete. GGBFS has been used effectively to control both ASR and external sulfate attack.

Silica Fume

Silica fume is the byproduct of induction arc furnaces used in the silicon metal and ferrosilicon alloy industries where quartz is reduced to silicon at temperatures up to 2000 °C (Mehta and Monteiro 1993). This produces SiO vapors that oxidize and condense to minute spherical, noncrystalline silica. The particle size is roughly two orders of magnitude smaller than fly ash, having an average diameter of 0.1 μm and a surface area of 20,000 to 25,000 m^2/kg, which is twice that of tobacco smoke (Kosmatka et al. 2002). It is highly pozzolanic, but its incredibly high surface area makes it difficult to handle, significantly increasing the water demand of the concrete unless water-reducing admixtures are used. Standard specifications for silica fume are provided in ASTM C 1240.

Due to its fine size, silica fume is able to pack very tightly into void spaces between cement and aggregate particles. It can thus be used to significantly reduce concrete permeability and therefore block the ingress of chlorides. For this reason, silica fume concrete has seen extensive use on bridge decks. Its highly pozzolanic nature also converts alkalis and CH into CSH, making the concrete less susceptible to chemical attack. Thus, it can be used to mitigate ASR and sulfate attack (ACI 1992; Kosmatka et al. 2002), although its effectiveness must be established through testing.

Silica fume has not been widely used in pavements because of problems encountered in handling and special concerns related to curing. It is also costly and is therefore unlikely to find widespread use in pavement applications, except under special circumstances.

Admixtures

A number of noncementitious admixtures can be added during proportioning or mixing to enhance the properties of freshly mixed and/or hardened concrete. Admixtures used in pavement construction in North America include air entrainers, water reducers, accelerators, retarders, corrosion inhibitors, noncementitious minerals, and ASR inhibitors. An excellent description of these various admixtures can be found in a number of sources (Kosmatka et al. 2002; Mehta and Monteiro 1993; Mindess and Young 1981). The following is a brief discussion specifically focused on the impact various admixtures have on concrete pavement durability.

Air-Entraining Admixtures

Air-entraining admixtures are specified and tested under ASTM C 260 and C 233, respectively. Air-entraining admixtures are added just prior to or during concrete mixing and can be used in place of or in conjunction with air-entraining cement. The entrained air voids protect the hardened concrete against freeze-thaw damage and deicer scaling. They also improve the workability of the fresh concrete, significantly reducing segregation and bleeding.

Air-entraining admixtures function by stabilizing bubbles formed in the fresh paste during mixing. This is accomplished through surface-active agents that concentrate at the interface between air and water, reducing the surface tension so the bubbles are stable. These surface-active agents are composed of molecules that are hydrophilic (water loving) at one end and hydrophobic (water fearing) at the other. Thus they align at the interface with their hydrophilic ends in the water and the hydrophobic ends in the air (Mindess and Young 1981). Typical compounds used as air entrainers include salts of wood resins (Vinsol resins), salts of sulfonated lignin, salts of petroleum acids, alkylbenzene sulfonates, and salts of sulfonated hydrocarbons among others.

The dosage rate for air-entraining admixtures is usually very small, on the order of 0.005 to 0.05 percent active ingredients by weight of cement. Thus they are normally diluted to assist in accurate batching (Mindess and Young 1981). The amount of entrained air required to protect concrete depends on the exposure level and the nominal maximum aggregate size. Recommended air contents for freeze-thaw resistant concrete from ACI (1992) are reproduced in table III-9. Concrete pavements subject to deicer application are considered to be in severe exposure conditions.

The air content of fresh concrete can be determined using ASTM C 173 or C 231. It is noted that air content alone does not ensure the adequacy of the air-void system, but that a good correlation between air content and freeze-thaw distress exists for air entrained concrete. The complete air-void system in hardened concrete can be assessed microscopically using procedures described in ASTM C 457.

Water-Reducing Admixtures

Water-reducing admixtures are added to reduce the quantity of mixing water required to produce concrete of a given consistency. This allows for a reduction in the *w/cm* while maintaining a desired slump, and thus has the beneficial effect of increasing strength and reducing permeability. A reduction in water content by 5 to 10 percent is obtainable through the use of

Table III-9. Recommended air contents for freeze-thaw distress-resistant concrete (ACI 1992).

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

[1] A reasonable tolerance for air content in field construction is ± 1.5 percent.
[2] Outdoor exposure in a cold climate where the concrete will be only occasionally exposed to moisture prior to freezing, and where no deicing salts will be used. Examples are certain exterior walls, beams, girders, and slabs not in direct contact with soil.
[3] Outdoor exposure in a cold climate where the concrete may be in almost continuous contact with moisture prior to freezing, or where deicing salts are used. Examples are pavements, bridge decks, sidewalks, and water tanks.
[4] These air contents apply to the whole as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 37.5 mm (1-1/2 in) is removed by handpicking or sieving and the air content is determined on the minus 37.5 mm (1-1/2 in) fraction of the mixture. (The field tolerance applies to this value.) From this the air content of the whole mixture is computed.

conventional water reducers that are specified under ASTM C 494 Type A. This class of water reducer typically will retard set, and accelerators are often added to offset this effect. If it is desired that the water reducer also acts as a retarder or accelerator, it can be specified under ASTM C 494 Type D and Type E, respectively. ASTM C 494 water reducers are typically composed of lignosulfates, hydroxylated carboxylic acids, or carbohydrates.

The effect of water reducers on the fresh concrete properties varies with the chemical composition of the admixture, the concrete temperature, cement composition and fineness, cement content, and the presence of other admixtures (Kosmatka et al. 2002). The effect on the air-void structure is unclear, with some sources reporting an improvement (Kosmatka et al. 2002) while others reporting possible adverse affect (Pigeon and Plateau 1995). Thus, the fresh and hardened concrete properties of mixtures containing water reducers should be thoroughly evaluated during design to determine the extent of detrimental interactions that may occur.

High range water reducers (ASTM C 494 Type F and G) can reduce water content by 12 to 30 percent, but their use is primarily restricted to applications where "flowing" concrete is desired ((Kosmatka et al. 2002). Also, the entrained air voids produced in concrete made with superplasticizers are often large, increasing the spacing factor, and on occasion instability in the air-void system may occur (Pigeon and Plateau 1995).

Accelerators

Accelerators, specified under ASTM C 494 Type C, are used to increase the rate of concrete strength development. Their use in concrete paving is generally restricted to fast track projects or cold weather construction where the rapid liberation of heat helps prevent freezing during early stages of hydration. Accelerating strength development can have negative effects on concrete durability, and thus should only be done when critical time constraints exist.

The most commonly used accelerator is calcium chloride as specified under ASTM D 98. This admixture is a very effective accelerator and is relatively inexpensive. But its use can contribute to MRD by increasing the corrosion of reinforcing steel, drying shrinkage stresses, scaling potential, potential for alkali–aggregate reactions, and susceptibility to sulfate attack. Therefore, the use of a calcium chloride as an accelerator in pavement applications is discouraged.

Other nonchloride, noncorrosive accelerators are available that contain compounds such as triethanolamine, sodium thiocyanate, calcium formate, calcium nitrite, and calcium nitrate. Their impact on concrete durability is not well documented, but it is very unlikely that durability will be improved (except for corrosion resistance for those that inhibit corrosion of embedded steel). It has been generally observed that concrete microstructure produced in rapidly setting concrete is coarser and composed of more soluble hydration products that are prone to chemical attack. It is therefore emphasized that accelerators should only be used when absolutely necessary, and that the designer/engineer understand that their use will likely have a negative impact on the long-term durability of the concrete.

Retarders

Retarders, specified under ASTM C 494 Type B, are used to delay the setting time of concrete. Common compounds used as retarders are lignin, borax, sugars, tartic acids, and salts. Most retarders also act as water reducers, and may also have some air entraining properties (Kosmatka et al. 2002). As with all admixtures, it is critical that the effect of the retarder on the fresh and hardened concrete properties be investigated during the mix design stage.

Corrosion Inhibitors

Corrosion inhibitors are admixtures specifically added to concrete to inhibit the corrosion of steel. Their use in pavement construction is quite limited, as alternative methods of protecting steel (e.g., the use of high quality, low permeability concrete, adequate cover, and epoxy coatings) have been used with relatively good success. But in pavement structures containing significant amounts of steel that will be subjected to frequent chloride applications, the use of a corrosion inhibitor might be considered.

The most common corrosion inhibitor is calcium nitrite, which also acts as an accelerator. A number of studies have verified its ability to prevent corrosion by reinforcing and stabilizing the passive film that protects steel (Kosmatka et al. 2002; ACI 1992). The dosage rate varies, but usually falls in the range of 26 to 110 ml/kg cement. It is unclear what other effects the addition of calcium nitrite has on concrete durability, but it is likely to result in an increase in CH formation in the paste and higher alkali levels, which in turn may adversely affect concrete durability. Other corrosion inhibitors include compounds such as sodium nitrite, sodium benxoate, certain phospates, flousilicates, or fluoaluminates. Corrosion inhibitors are relatively expensive, increasing the cost of a concrete structure by approximately 8 percent (Mehta 1997).

Noncementitious Mineral Admixtures

A number of noncementitious mineral admixtures could be added to concrete. Pulverized limestone is one that may improve durability of pavements, having some benefit in reducing ASR. It is noted that it is not as effective as pozzolans. Typically noncementitous mineral admixtures are added to improve workability caused by lack of fines. As such, the addition of the mineral admixture for this purpose is considered a mix design issue.

ASR Inhibitors

Lithium compounds are the best known ASR inhibitors. Their effectiveness in preventing deleterious ASR depends on the lithium compound used, addition rate, aggregate reactivity and cement alkalinity (TRB 1999). The most effective and easiest to handle lithium compound is lithium nitrate ($LiNO_3$). Other lithium compounds that have been investigated include lithium hydroxide ($LiOH$), lithium hydroxide monhydrate ($LiNO_3 \cdot H_2O$) and lithium carbonate (Li_2CO_3) (AASHTO 2000). When lithium hydroxide is added to concrete, it forms minimally expansive lithium-bearing ASR gel, which is generally not damaging to the concrete (Farny and Kosmatka 1997). It is noted that ASTM C 1260 cannot be used to assess the effectiveness of lithium compounds (Farny and Kosmatka 1997). The recommended use of lithium compounds for mitigation of ASR is presented in the AASHTO Guide Specifications (AASHTO 2000).

Water

Almost any potable water that has no pronounced taste or odor can be used to make concrete. ASTM C 94 provides guidance regarding development of compressive strength and time of set as reproduced below in table III-10. It is noted that the criteria have no provision regarding long-term durability of the concrete. It is therefore recommended that if the water is of questionable quality, the chemical requirements provided in ASTM C 94 for wash water be applied. These are reproduced in table III-11. From a concrete durability perspective, alkali, chloride, and sulfate contents must be considered when selecting mixing water.

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

To this point, the guideline has considered the properties of the individual constituent materials of a concrete mixture. Once the materials are selected, they must be economically combined to form concrete that is workable and easy to consolidate, develops adequate strength, and has long-term durability. The PCA (Kosmatka et al. 2002) and the ACI (1991) both present acceptable methods for proportioning concrete mixtures. The discussion in the following section focuses exclusively on elements of the proportioning and mix design process that have a direct bearing on enhancing concrete durability.

The key to durable concrete is the use of high quality materials arranged in a dense, relatively impermeable matrix. The selected aggregates should be strong and not be reactive or susceptible to freeze-thaw or moisture damage within the environment in which they serve. If this condition is met, the hydrated cement paste is the weak link in the matrix, and thus the goal of proportioning aggregates is to ensure that they occupy as high a percentage of the concrete volume as possible. The hardened paste microstructure should be free of microcracking, possess

Table III-10. ASTM C 94: Acceptance criteria for questionable water supplies.

	Limits	Test Method
Compressive strength, minimum percentage of control at 7 days	90	ASTM C 109[1]
Time of set, deviation from control, hr:min.	from 1:00 earlier to 1:30 later	ASTM C 191[1]

[1] Comparisons should be based on fixed proportions and the same volume of test water compared to control mix using city water or distilled water.

Table III-11. ASTM C 94: Optional chemical limits for wash water.

Chemical	Maximum Concentration[1] (ppm)	Test Method[2]
Chloride, as Cl		ASTM D 512
Prestressed concrete or concrete in bridge decks	500[3]	
Other reinforced concrete in moist environments or containing aluminum embedments or dissimilar metals or with stay-in-place galvanized metal forms	1,000[3]	
Sulfates, as SO_4	3,000	ASTM D 516
Alkalies, as $(Na_2O + 0.658\ K_2O)$	600	
Total solids	50,000	AASHTO T 26

[1] Wash water reused as mixing water in concrete can exceed the listed concentrations of chloride and sulfate if it can be shown that the concentrations calculated in the total mixing water, including mixing water on the aggregates and other sources, does not exceed the stated limits.

[2] Other test methods that have been demonstrated to yield comparable results can be used.

[3] For conditions allowing the use of $CaCl_2$ accelerator as an admixture, the chloride limitation may be waived by the purchaser.

a fine pore structure with an adequate air-void system to protect it from freeze-thaw damage, and be composed of stable and relatively insoluble hydration products. Steel must be protected with adequate cover, and the concrete must be properly cured and free of drying shrinkage cracking.

Specific requirements for the selection of concrete mixture characteristics, aggregate grading, and tests for fresh and hardened concrete are discussed below.

Selection of Concrete Mixture Characteristics

The first step in the proportioning process is to select mixture characteristics consistent with the production of durable pavement concrete. The *w/cm* is probably the most important factor in this regard. It is recommended that the *w/cm* not exceed 0.45 for pavements that will experience freezing and thawing in a moist environment and will be exposed to deicer applications. In the same environment, reinforced concrete pavements should be constructed with a maximum *w/cm* of 0.40 unless 12.5 mm of additional concrete cover is used (ACI 1991). If external sources of sulfate are present, the recommendations presented in table III-12 should be followed (ACI 1992). When the environment is such that durability is not a controlling factor, the *w/cm* can be selected based on strength requirements (TRB 1999), but it is generally not recommended that a *w/cm* exceeding 0.50 be used in pavement concrete.

Table III-12. Recommendations for normal weight concrete subject to sulfate attack (ACI 1992).

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

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

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

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

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

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

In addition to the *w/cm*, it is sometimes recommended that a minimum cement content of 335 kg/m^3 be used in environments subjected to severe freeze-thaw cycling, deicer applications, or severe sulfate exposure (Kosmatka et al. 2002). The TRB Circular entitled *Durability of Concrete*, indicates that the rationale for such requirements may be invalid, advocating the use of performance-based specifications instead (TRB 1999).

Commonly, a 28-day compressive strength or modulus of rupture is specified for paving concrete as a general measure of concrete quality. As addressed previously, strength at 56 days or 90 days might be considered for the production of durable pavement concrete, as it better allows for the use of fly ash and GGBFS as an addition or replacement for portland cement.

Slump requirements for slip-formed paving concrete typically fall in the range of 25 to 75 mm. Concrete consistency should be such that the mix is not difficult to consolidate and is not prone to segregation under the action of internal vibration. It also must be stiff enough that sloughing of the paving edges is not a problem. Table III-9 should be used to select an appropriate entrained-air content based on the exposure condition and the nominal maximum aggregate size.

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

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

Aggregate Grading

The aggregate characteristics that have the largest impact on proportioning are the particle size distribution (grading) and the nature of the particles (shape, angularity, porosity, and surface texture) (Kosmatka et al. 2002). These two parameters affect both the handling of fresh concrete and the properties of hardened concrete.

Traditional grading methods presented in standard mix design procedures are based on the blending of a coarse aggregate and a fine aggregate grading as presented in ASTM C 33. Both the ACI and PCA procedures recommend that the largest practical aggregate size be used for job conditions. This will maximize economy as the increased aggregate size will minimize void space and thus minimize paste requirements. Larger size aggregates also have the benefit of increasing aggregate interlock across cracks and joints, thus improving pavement structural performance, while reducing drying shrinkage. In each design procedure, the maximum aggregate size for the coarse aggregate is selected based on the slab thickness and the spacing between reinforcing bars. For pavements, both ACI and PCA recommend that the maximum aggregate size should be less than one-third the slab thickness and three-quarters the free space between reinforcing bars or reinforcing bars and formwork. For example, a 200-mm-thick plain jointed concrete pavement (JCP) could, under these guidelines, use a maximum aggregate size up to 66 mm.

The maximum aggregate size typically used in modern pavement construction is significantly smaller than the maximum permissible under the standard mix design practices. In addition, the maximum size of the aggregate used in paving concrete has decreased in recent years, particularly in the northern midwestern States, where aggregate freeze-thaw deterioration has been a problem. For example, the Michigan Department of Transportation (MDOT) commonly uses a 19-mm maximum aggregate size in pavement concrete, whereas in the mid-1970's it was not unusual to use 37.5- to 50-mm maximum coarse aggregate size. The movement toward smaller maximum coarse aggregate size was in response to poor freeze-thaw durability of some aggregate sources. But over time, the practice of using smaller aggregate has become standard practice for all pavement concrete regardless of the aggregate's durability. MDOT and others are currently evaluating the use of larger aggregate mixtures to improve aggregate interlock and minimize drying shrinkage.

Using a larger maximum aggregate size is one method to increase the proportion of aggregate in the mixture. A second method is to achieve a more continuous aggregate grading than is commonly used in traditional mix design methods. Strategic Highway Research Program (SHRP) research recommended such a modification to ASTM C 33 because there is considerable variation within the packing densities of aggregate specified according to the standard (Roy et al. 1993b). These variations could lead to noticeably different packing densities for aggregate combinations having the same maximum aggregate size. Also, it was noticed that some gradations that were acceptable under ASTM C 33 produce mixes with little tolerance for aggregate proportioning during construction.

The U.S. Air Force believes that aggregate gradation has been a major factor in joint spalling, with poorly proportioned mixtures being more difficult to construct and exhibiting early distress manifestations (Muszynski and LaFrenz 1996; Muszynski et al. 1997). Consequently, they have changed their specification for grading aggregates in concrete mixtures, considering both the coarseness and workability of the gradation. Aggregate is no longer just considered to be either fine or coarse, but instead is viewed as a single combined aggregate blend. This approach is echoed in the TRB Circular on concrete durability, which states that a lack of mid-sized aggregate (around 9.5 mm size) results in concrete with high shrinkage, high water demand, and poor workability (TRB 1999). Blending of multiple aggregate sources can be used to address this problem (Shilstone 1990).

Mehta (1997) states that it is desirable from the perspective of durability to maximize the aggregate content and minimize the paste content to promote the watertightness of the concrete. It is believed that the use of the largest maximum aggregate size (considering freeze-thaw durability of the aggregate) with a continuous grading may provide an opportunity to meet these goals.

Tests for Hardened Concrete Relating to Durability

A number of standard tests are available that can be used to establish the susceptibility of concrete to certain MRDs. Unfortunately, many of these standards are designed to test only a single mix constituent, such as the cement or the aggregate. For example, ASTM C 1293 *Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction* is designed to only test the aggregate under consideration for ASR susceptibility, and not how the cementitious materials or other admixtures affect the outcome. Likewise, a test such as ASTM C 1012 *Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution* will test the sulfate resistance of the cementitious material, but not in combination with the other mix components. Additionally, many of these tests do not adequately model expected field exposure conditions, and thus may not reflect actual field performance. Finally, many of the tests used to evaluate MRD are long term and are thus not ideally suited to mix design.

Even with these limitations, a few standard test methods may be useful in assessing the potential durability of pavement concrete. It is assumed that the constituent materials have been screened as described in this guideline. This is particularly important for aggregate screening, as it is used to prevent such MRD types as aggregate freeze-thaw deterioration, ASR, and ACR. It is also assumed that common tests on fresh concrete, such as slump, air content, and strength testing of hardened concrete, are conducted. When appropriate, the actual job mix formula must be used with the mix constituents including aggregate, cement, admixtures, and water. The duration of the mixing cycle in the laboratory should be consistent with that anticipated during construction.

AASHTO PTP 34-99: Proposed Standard Method of Test for Restrained Drying Shrinkage

There is a standard test method currently under review for the determination of drying shrinkage. The test, specified in AASHTO TP 34-99, helps distinguish concrete that will undergo excessive drying shrinkage, which in turn can result in cracking and loss of watertightness. In this ring test, specimens are molded in two layers surrounding a steel ring. After 24 h the outer mold is removed, exposing the concrete surface. The top and bottom faces of the concrete rings are covered with silicone rubber sealer to prevent any moisture loss other than through the circumferential area. A steel ring inside the concrete specimen restrains the concrete specimen as it tries to shrink. This restraint will develop internal tangential tensile stresses, which will cause the concrete to crack once its tensile strength is exceeded (Kraai 1985). The time to cracking and the width and length (area) of these cracks represent the concrete's resistance to drying shrinkage cracking.

ASTM C 457: Practice for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

This test is commonly used to assess the air-void distribution in hardened concrete. Most often used as a diagnostic test, ASTM C 457 could be applied during the mix design process to assess the adequacy of the air-void system. It can also be conducted during construction on field cores. This type of evaluation will assist in determining if interactions between the various constituents has led to changes in the anticipated air-void system. Test results could also be used as a baseline to determine if field construction is producing an air-void system consistently different than that produced in the lab. Finally, results of ASTM C 457 could be used to assess the potential of secondary ettringite formation (SEF) by monitoring the filling of voids with ettringite over time, as it provides a baseline measurement of the unfilled air-void system.

ASTM C 666: Test Method for Resistance of Concrete to Rapid Freezing and Thawing

This test has been previously described in this guideline. In the evaluation of aggregate, it is typical to make a standard mix (specified cement type, *w/c*, and air content) to compare aggregate performance. But it is known that the full interaction of constituent materials can not be easily anticipated, and thus freeze-thaw testing of concrete specimens made with the job mix formula and materials may be useful in identifying potential problems. Unfortunately, this is a long-term test, requiring upwards of 3 months to conduct. It therefore should be reserved for use when marginal aggregates are being considered.

ASTM C 672: Test Method for Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals

ASTM C 672 is the most common test used to investigate the scaling potential of concrete. In this test, a $CaCl_2$ solution (other chemical deicers may be used) is ponded on the surface of rectangular concrete specimens that are then subjected to freeze-thaw cycling. The specimens are placed in a freezer (-17.8°C) for 16 to 18 h and then manually removed to a thawing environment for 6 to 8 h. A surface layer of the water/salt solution is maintained at all times. A visual inspection is made at 5, 10, 15, 20, 25, and 50 cycles. The concrete is rated on a scale of 0 to 5 (0 is for concrete surfaces showing no sign of scaling and 5 is for a surface that is severely scaled with coarse aggregate visible over the entire surface). The subjectivity inherent in the rating scale is one problem with this test. As a result, it is becoming common for researchers to measure the mass of scaled material to gain a more objective measure of scaling resistance.

This test is most applicable when the concrete pavement will be subjected to deicer applications and supplementary cementitious materials are being used. Concrete made with fly ash and GGBFS have exhibited variable deicer scaling resistance, and therefore mixtures containing these materials should be lab tested if field experience does not exist. Control samples of mixes with known scaling resistance should also be tested, as the results of this test are qualitative.

ASTM C 1202: Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

This test provides excellent correlation to the results obtained by AASHTO T 259, *Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration,* for a wide variety of concrete types and qualities (Perenchio 1994). One exception is concrete made with silica fume because the active pozzolanic nature of the material makes it seem more impermeable to chloride ion penetration than it really is. Another exception is concrete made with certain admixtures that affect the electrical conductivity of the mixture, including calcium chloride or calcium nitrite. Regardless, the rapidity, ease of use, and reliability make this test very attractive when investigating not only the chloride ion permeability characteristics of concrete, but also to assess permeability in general. Table III-13 can be used to make a general assessment of the chloride ion permeability of the concrete tested using this test method (ASTM C 1202). It is noted that the assessment is not specific, but instead the chloride permeability is assigned a qualitative rating. Pavement concrete should have a qualitative permeability not greater than low, and in aggressive environments (exposure to chemical deicers, sulfates, etc.), a qualitative rating of very low or less is desirable.

Table III-13. Chloride ion penetrability based on charge passed (AASHTO T 277).

Charge Passed (coulombs)	Chloride Ion Permeability
>4,000	High
2,000-4,000	Moderate
1,000-2,000	Low
100-1,000	Very Low
<100	Negligible

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

ASTM C 642: Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete

Porosity is a measure of the proportion of the total volume of pore space in concrete irrespective of the interconnectivity of the pores (Neville 1996). Absorption tests can be used to measure porosity, but the degree of interconnectivity between the pores influences the measured absorptivity. Thus, although porosity and absorptivity are commonly correlated, there is not a direct relationship. A variety of techniques are used for determining the absorption rate of concrete. One common test is ASTM C 642, *Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete,* which entails drying a concrete specimen at 100 to 110°C and then immersing it in water at 21°C for at least 48 h. This type of test is commonly used as a quality control test for precast members.

Sorptivity Test

As a result of shortcomings in common absorption testing procedures (e.g., ASTM C 642), there has been movement toward a type of testing known as sorptivity testing. Sorptivity testing measures the rate of absorption by capillary suction of water into the concrete (Neville 1996). Generally, it is too difficult to mathematically model this flow in all but a single direction, and thus sorptivity tests are configured to establish one directional flow into the specimen (Stanish et al. 2000). Sorptivity tests typically require that the sample be at a standard moisture content before testing is begun. The benefits of sorptivity testing are reduced time, low cost of equipment, and simplicity of procedure. The proposed ASTM standard test for sorptivity requires only a scale, a stopwatch, and a shallow pan of water (Stanish et al. 2000). One attractive feature of this approach is that the sample is conditioned for 7 days, with the temperature never exceeding 50 °C. This is important since damage to the concrete microstructure can result at the higher drying temperatures (100 °C or higher) recommended in other test methods, thus biasing the test results. Data reduction includes plotting the gain in mass per unit area over the density of water versus the square root of elapsed time, with the slope of the best-fit line being reported as sorptivity.

Testing Needs

As mentioned, a limitation of many of the standard tests is that they are designed to test only one element of the mix design. For example, in ASTM C 1260 or C 1293, aggregate is tested under controlled, severe exposure conditions. Aggregates that pass either of these tests will likely not incur ASR in the field, but for marginal aggregate, the results are less certain. If marginal aggregates are to be used, it is recommended that low alkali cement and/or a low calcium fly ash or GGBFS be added. Unfortunately, there is no rapid standard test method available that can be used to judge the effectiveness of these admixtures, primarily due to the nature of the ASR reaction, which in some cases can take years to appear.

Efforts are under way to address this limitation. For example, the proposed AASHTO Guide Specification for ASR (AASHTO 2000) recommends using both ASTM C 1260 and C 1293, as well as ASTM C 441 (*Test Method for Effectiveness of Mineral Admixtures in Preventing Excessive Expansion of Concrete Due to Alkali-Aggregate Reaction*) to test the effectiveness of proposed mitigation. But for other MRD types, the implementation of new test methods is not imminent. Gress (1997) laments the fact that a rapid standard test does not exist for the evaluation of internal sulfate attack in field-proportioned concrete (although the prevalence of this distress in paving concrete has yet to be determined). Currently, test methods employed by researchers primarily focus on the expansion of the cementitious materials and can take years to complete. Considerable research is needed in this area before a standard test will be available. Until then, screening constituent materials offers the best alternative to ensure concrete durability.

Construction Considerations for Preventing MRD

Construction can have a very important impact on the occurrence of MRD. As stated, durable concrete must be relatively watertight and nonreactive. From a construction perspective, this means that it must be properly batched, mixed, consolidated, and cured under conditions that will produce little microcracking and stable hydration products. Construction practices that have a direct impact on batching, mixing, consolidation, and curing include ambient and seasonal conditions, batching, duration of the mixing cycle, method of consolidation, steel placement, finishing, and curing. The following discussion focuses exclusively on those elements of the construction sequence that have a direct bearing on concrete durability. It is assumed that the concrete is well designed and workable, and that standard concrete construction practices are being followed.

Ambient and Seasonal Conditions

Ambient and seasonal conditions can play an important role in the durability of concrete through their influence on the development of drying shrinkage cracking, excessive heat of hydration, and inadequate curing prior to deicer application. Hot-weather and cold-weather construction practice as described in *Design and Control of Concrete Mixtures* (Kosmatka et al. 2002) and elsewhere must be followed. It is imperative that plastic and drying shrinkage cracking be avoided to maintain the integrity of the concrete surface. This can only be accomplished if an awareness of the relationship between relative humidity, temperature, and wind exists and is acted upon by the construction team. Curing practices should be established that are flexible enough to respond to changes in climatic conditions. A tool that is available to assist in predicting early-age climatic effects is the software package HIPERPAV (McCullough and Rasmusssen 1999).

Hot ambient temperatures potentially may affect the development of MRD. In addition to increased problems with plastic and drying shrinkage cracking, air loss, and workability, it is known that relatively high concrete temperatures can occur during curing if a high cement content mix is placed in the late morning on a very hot day. Gress (1997) cites a number of studies that found pavement temperatures exceeded 45°C during curing. It is feasible that if precautions are not taken, the internal temperature of a thick slab made with a rich, Type III cement mixture placed during a hot summer day may approach 60°C, a temperature at which DEF may occur. It is therefore necessary to control the mixture temperature during paving to avoid detrimental heat build-up within the first 24 h.

Batching

Batching is obviously a very important construction factor that contributes to the durability of the concrete. If the mixture is improperly batched, either through improper addition rate or the order in which materials are added, undesirable consequences may occur. One of the most important aspects is the control of water, since it directly impacts the *w/cm*. The water content of the aggregates must be monitored and accounted for, and provisions must be made for the water remaining in the mixer after washing (TRC 1999). Also, the accurate batching of admixtures can be critical with respect to durability.

Mixing Cycle

As speed of construction has increased, the use of shorter mixing cycles has been investigated by a number of SHAs. This shorter mixing cycle addresses the increased material demand resulting from the use of larger, more powerful pavers. Unfortunately, a decrease in the length of the mixing cycle affects the homogeneity of the resulting concrete, with balling, clumping, and sand streaking becoming more prevalent (Gress 1997). In addition, air-entraining admixtures require a certain minimum amount of mixing to disperse the air voids uniformly throughout the mixture. If this does not happen, freeze-thaw damage in the paste may occur. Thus, any shortening of the mixing cycle should be carefully investigated as it may have a detrimental impact on the durability of the concrete.

Consolidation

The goal of consolidation of paving concrete is the removal of entrapped air while avoiding segregation and disruption of the entrained air system. The concrete mixture properties are key to good consolidation. Assuming that a high quality, properly graded, workable mixture is used, it can be consolidated through mechanical means including both internal and external vibration. Internal vibration is applied through immersion-type vibrators typically located after the strike-off in the paver. Important factors to consider are the frequency of vibration, amplitude, and the speed of paving. Frequency is typically set between 7,000 and 12,000 vpm, although a recent study recommends fixing the frequency at 10,000 vpm (Gress 1997). This same study recommends weekly calibration checks on the vibrators, a negative (down from horizontal) vibrator slope of 30^{0} or to one-half the pavement depth, and a fixed, uniform paver speed. Other sources state that 8,000 vpm is more typical. In any event, it is critical that the vibration is set to consolidate the concrete without segregation or disruption to the entrained air-void system (TRB 1999).

It is emphasized that under-vibration can result in poor consolidation and over-vibration may lead to segregation and the elimination of the air-void system in the immediate vicinity of the internal vibrator. It has been noted on a number of MRD affected projects that the vibrator trails are visible on the pavement surface and appear to be an initiation point for cracking, spalling, and scaling. In some instances, petrographic analysis has revealed that the air content in the path of vibration is significantly less than that between the vibrators.

When constructing concrete pavements, considerable attention must be directed to produce a mix that has consistent consolidation characteristics. Early in the construction of the project, a paving rate and vibrator frequency must be established that adequately consolidates the concrete without harmful segregation or disruption to the air-void system. If the mix is found to be harsh and difficult to place, construction should be stopped until the problem is corrected. Too often, difficulties in placement are not discovered until a pavement begins to suffer early signs of deterioration at which point little can be done.

Steel Placement

The placement of embedded steel in concrete pavement has a direct bearing on the potential for corrosion. Steel used at joints, such as dowel and tie bars, are typically placed at mid-depth, thus concrete cover is typically not an issue. But in these applications, the joint provides a direct route for chloride ion and moisture ingress, and thus these bars must be protected with a durable coating to avoid corrosion. Epoxy is the most commonly used protective coating, although the cladding of dowels with plastic and stainless steel has also been used by some agencies.

For reinforcing steel used in jointed reinforced concrete pavement (JRCP) or in CRCP, adequate concrete cover is essential to prevent corrosion. At least 50 mm of quality concrete cover is required in environments where chloride-based deicers are used. To ensure that at least 90 to 95 percent of the reinforcing steel is adequately protected, it is recommended that a design cover of 65 mm be used. During construction, great care must be exercised to avoid areas of "high steel" that become particularly susceptible to corrosion.

Finishing

In slipform paving, mechanical floating is commonly used immediately following the vibrating pan to embed large aggregate particles, correct small surface imperfections, and "close the surface." If the entire slipform paving operation proceeded smoothly, no other finishing other than texturing may be required. But in most cases, additional finishing is needed to correct surface imperfections.

After paving, the surface is checked using a 3- to 4-m straight edge. Surface imperfections are commonly corrected using a hand-operated float. It is common that little bleed water occurs with stiff, slipformed concrete, but under certain conditions, it may be present. It is emphasized that finishing should not be conducted when bleed water is present, as there is a danger that either bleed water will become trapped beneath the concrete surface, creating a plane of weakness, or air content in the surface layer will be reduced. In either case, scaling of the surface may occur due to freeze-thaw damage, particularly if chemical deicers are used.

Finishers commonly desire to add more water to the surface to assist in the finishing operation. This tendency must be resisted as working water into the concrete surface decreases the *w/cm* and air content at the surface, leading to spalling and/or scaling. If the mix is difficult to work without water being added during finishing, the mixture design should be reviewed.

On occasion, a paving edge may slump, requiring correction. In this case, it should be carefully rebuilt with added concrete, working against a bulkhead placed against the slab edge. Care must be taken to avoid over-working or over-finishing the repair. Edge slumping should not be common, and if it is occurring with regularity, the entire paving operation must be evaluated.

Texturing the surface is accomplished most often through the use of a burlap drag and tining. The burlap drag must be moist, but not excessively so. Water should be carefully applied to the burlap in such quantity that there is not a trail of bubbles observed behind the drag. Following the drag, it is common to tine the surface using a tine texturing machine, imparting macrotexture to enhance surface friction.

Curing

In an ideal world, concrete would be cured in such a way that the presence of mixing water is maintained. This requires the continued application of water either through ponding, fogging, or covering the concrete with a saturated cloth such as burlap. Unfortunately, wet curing is not practical for most paving projects. Instead, the most common curing method is to minimize the loss of mixing water through the application of a membrane-forming curing compound as specified under ASTM C 309. The effectiveness of curing compounds can be assessed through ASTM C 156.

The curing compound should be applied immediately following texturing. Any delay, particularly during hot, windy conditions, can cause significant harm to the concrete resulting in plastic shrinkage cracking. Although these cracks are small and isolated to the concrete surface, they impair the watertightness, providing access for potentially deleterious compounds into the concrete structure.

Great care should be exercised in the application of curing compounds. The application must be uniform and of sufficient quantity to ensure that the surface is completely sealed. On large paving projects, power-driven spray equipment should be used. It is highly recommended that two coats be applied to ensure complete coverage (Kosmatka et al. 2002).

There are concerns that some curing compounds are not effective. SHAs are highly encouraged to test curing compounds to verify their moisture-retention properties. Water-based curing compounds should not be "watered down" during construction.

Different concrete mixtures require different lengths of curing prior to opening to traffic. This must be carefully considered when concrete mixtures containing fly ash or GGBFS are used. Temperature also has a major effect on the required length of curing, and the use of maturity concepts to determine appropriate opening times is highly encouraged.

In addition to strength gain required for load carrying capacity, newly placed concrete also requires an air-drying period of one month before it is subjected to deicer applications. Research has found that this will add to the scaling resistance of the concrete by reducing the degree of saturation.

Controlling Specific Types of MRD

The previous discussion focuses on elements of material selection, mix design, and construction that have a direct bearing on the formation of MRD in concrete pavements. The approach taken is the selection of inherently durable materials, combined and constructed to create concrete that is relatively impermeable and physically and chemically stable. In this section of the guideline, approaches regarding the prevention of specific MRD types are presented. A SHA that has had specific problems regarding one or more MRD types can use this section to examine what preventive strategies are available to avoid future occurrences. Also, during material screening, some of the constituents may have been found to be marginal and mitigation was recommended. In either case, the information in the following sections can be used to help avoid the occurrence of specific MRD.

Paste Freeze-Thaw Deterioration

The only practical technique available to avoid paste freeze-thaw damage in a freeze-thaw environment is to entrain air voids of the proper size and spacing 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.

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, if a SHA has been suffering paste freeze-thaw damage, the air-void system in hardened concrete should be assessed microscopically during mix design and as part of the construction process using procedures described in ASTM C 457. 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, and therefore is a good investment if problems with this MRD have been encountered in the past. The use of an automated air-void analysis system can significantly reduce the time and labor required to conduct this testing.

If an SHA is also concerned with aggregate freeze-thaw durability, ASTM C 666 (Method A) can be conducted on concrete beams made using the materials and mix design selected for the project. In this way, the overall freeze-thaw durability of the mixture can be assessed. Additionally, if failures occur, procedures described in the second guideline in this series, *Guideline II — Laboratory Testing, Data Analysis, and Interpretation Procedures for Distressed Concrete Pavements*, can be used to determine the failure mechanism.

Deicer Scaling/Deterioration

In general, deicer scaling is not a concern for high quality PCC that has been properly constructed. Kosmatka et al. (2002) report that air-entrained concrete containing fly ash has similar freeze-thaw durability to concrete made with portland cement as the sole binder as long as the same compressive strength, air-void system, and curing are obtained. But deicer scaling is more of a concern with concrete containing supplementary cementitious/pozzolanic admixtures and therefore when using fly ash or GGBFS in concrete that will be subjected to chemical deicers, the scaling resistance should be assessed using ASTM C 672. The test should be conducted on concrete made using the actual job mix formula and materials. In general, it has been found that using a high cementitious content and a low *w/cm* (0.45 or less) will result in good scaling resistance (assuming proper finishing and curing).

In addition, if fly ash or GGBFS are being used, a longer than normal curing period prior to deicer application must be implemented, favoring spring/early summer construction over fall.

Aggregate Freeze-Thaw Deterioration

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

Therefore, if a marginal aggregate source is being considered for use, it must undergo benefaction to reduce the susceptibility to this MRD as illustrated in figure III-3. To this end, three methods have been used with variable success. The first method attempts to separate susceptible aggregate from nonsusceptible aggregate using the aggregate bulk specific gravity. It has been found in some cases that aggregates having lower bulk specific gravities were less resistant to freeze-thaw deterioration. 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 considered to be more resistant to freeze-thaw deterioration. Although this method is workable, it is not considered to be the most effective benefaction method.

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

The third method is to reduce the maximum aggregate size below the critical size needed to cause damage. This method has become standard practice in many States and overall has been effective in reducing the incidence of aggregate freeze-thaw deterioration. The degree that the aggregate size must be reduced varies with the aggregate type. In many instances, the maximum aggregate size has been reduced to 19.5 mm, and in some cases as small as 12.5 mm. Because of the ease of applying such criteria, it has become common practice in some States to specify mixtures having small maximum aggregate size even if the aggregate is not freeze-thaw susceptible. This practice, although seemingly effective in reducing aggregate freeze-thaw deterioration, has led to a significant increase in the paste requirement for pavement concrete and has compromised the structural integrity of cracks and joints relying on aggregate interlock for load transfer.

With the desire to use the largest maximum aggregate size possible, 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.

Regardless of the benefaction method employed, the effectiveness must be tested in the laboratory prior to construction. It is believed that the test method must confine the aggregate in a concrete matrix, such as in ASTM C 666 (Method A) and ASTM C 671. Either test requires expensive equipment and considerable time. But 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.

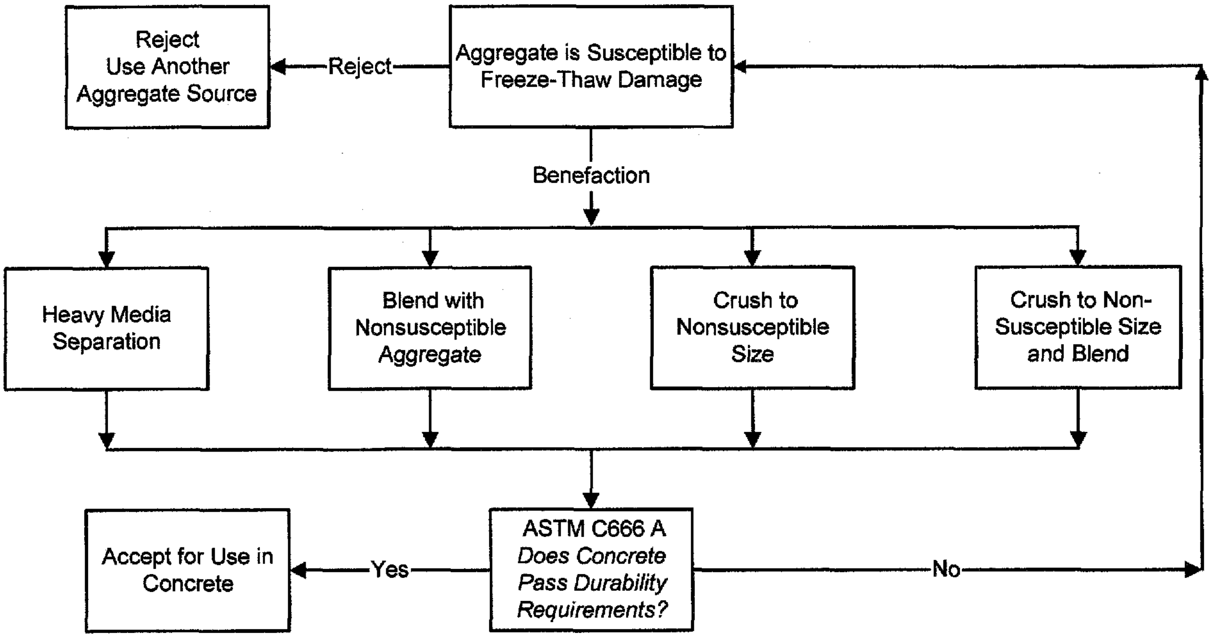


Figure III-3. Benefaction techniques for mitigation of aggregate freeze-thaw deterioration.

Alkali–Silica Reaction

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

The Proposed AASHTO Guide Specifications (AASHTO 2000) advance a slightly different approach, in which an ASR-susceptible aggregate is either rejected or mitigated. Mitigation is absolute in the sense that mixture proportions must be altered until the prevention criteria are met. There is no consideration for level of prevention required (based on risk level and design life). Instead the proposed standard adopts a "zero-tolerance" approach for deleterious ASR, regardless of the pavement's location or level of service. Thus, concrete used in a local road in a desert would have to meet the same prevention criteria as that going into a 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 is 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).

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, slag, 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 mix (Gress 1997). Gress (1997) 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. Gress (1997) further states that in areas with high ambient temperatures and highly reactive aggregate, such as in many southwestern States, 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 GGBFS.
- 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, GGBFS, 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).

Another mitigation technique that is showing increasing promise is the use of lithium-based additives. As discussed, 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 the 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 GGBFS are effective in reducing the ASR, but they note that this test cannot be used to judge the effectiveness of reducing total alkalinity or adding lithium compounds to the mixture. In addition, it is noted that the mechanism responsible for the reduction in expansion that occurs in either ASTM C 441 or C 1260 when used to test the effectiveness of GGBFS and/or a pozzolan might not be the same as would occur in the field. This is because the test duration of 14 days is too short for the slag and/or pozzolan to fully react (ACI 1998). In the Canadian standards, it is recommended that a 2-year test be run in accordance with ASTM C 1293 to 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. An SHA confronted with an ASR problem should consult the most recent research findings to assist in preventing ASR in future pavement construction.

Alkali–Carbonate Reactions

The surest way to prevent ACR is to screen aggregates prior to use in accordance with the previously described recommendations, rejecting those that are potentially susceptible to ACR. If potentially reactive aggregate must be used, it is possible to blend it with non-reactive aggregate to limit the potential for deleterious ACR. Farny and Kosmatka (1997) recommend that the reactive aggregate should be limited to 20 percent by volume if either the coarse aggregate or the fine aggregate is reactive. If both the coarse and the fine aggregate are reactive, then only 15 percent by volume of the aggregate used should be reactive. The maximum size of the aggregate also affects its reactivity. The larger the maximum size of the aggregate, the greater the potential that it is reactive, therefore the smallest feasible size of reactive aggregate should be used (ACI 1998).

The use of low alkali cements is far less effective in controlling ACR than ASR. Alkalies are regenerated through the alkali–carbonate reactions, and even cement alkali content as low as 0.40 percent can still lead to initiation of this distress (Farny and Kosmatka 1997). This is in contrast to literature cited in ACI 201.2R-26 (1992), which suggests that the use of low alkali cement will prevent harmful expansion in most cases. The most important cement parameter to consider when potential ACR susceptibility exists is to minimize the total alkali content per unit volume of concrete, and not focus exclusively on the alkali content of the cement (Ozol 1994). Pozzolans are also not effective in controlling ACR. However, preliminary evidence suggests that lithium-based admixtures appear to control ACR although little research has been performed to support this finding. Thus the only method currently available to prevent ACR is to eliminate or significantly reduce the amount of reactive aggregate.

External Sulfate Attack

The need for high quality, impermeable concrete is a prerequisite for concrete resistance to external sulfate attack. 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 external sulfate attack. ACI (1992) makes specific recommendations regarding not only the selection of the *w/cm,* but also the type of cement to be used in an aggressive sulfate environment. These recommendations were previously presented in table III-12. In moderate to severe exposure levels, a Type II or V cement is required. Cement low in tricalcium aluminate (C_3A) should be used if sulfate attack is anticipated, and some researchers state that the aluminoferrite phase (C_4AF) of portland cement should also be limited. For this reason, the Type V cement has a maximum calculated C_3A content of 5 percent and a combined $C_4AF + 2C_3A$ content that does not exceed 25 percent.

DePuy (1994) reports that using a 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. 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 a Type VP or VS cement be considered. Supersulfated slag cements, if available, are also an option.

Class F fly ash is generally found to be beneficial to sulfate resistance, whereas Class C fly ash may actually be detrimental. For these reasons, 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.

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

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 GGBFS with Type V cement, the combination to be used should be tested with an accelerated testing procedure. This is because the low alkali content of Type V cements may not activate the pozzolanic ingredients in the blended cements (ACI 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.

Internal Sulfate Attack

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

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

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

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. It is thus not thought to be a reliable indicator of DEF potential. Gress (1997) suggests that a new test method needs to be developed where heat cured mortar bars are made with the proposed constituent materials and length changes are measured over time.

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

Another potential source of slowly soluble sulfate is the aggregate. Sulfates having very low solubility in water, such as barium and lead sulfate (which are common in some aggregates) are relatively harmless. But some aggregates may contain moderately soluble sources of sulfate that may be problematic. For instance, work published by Peterson et al. (1999) found that calcium sulfide in slag coarse aggregate was soluble at pH levels common in concrete. A petrographic analysis of the aggregate, in accordance with ASTM C 295, should be used to identify potentially problematic sources of aggregate sulfates.

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

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

Corrosion of Embedded Steel

Chapter 3 in ACI 222R provides an excellent description of how to protect new concrete construction against corrosion. For pavement construction, the use of protective coatings on load transfer devices and adequate cover over reinforcing steel should be all that is required if high quality concrete is properly constructed.

The permeability of the concrete to chloride ion penetration is an important factor in determining whether the cover is adequate. This can be measured in the laboratory using ASTM C 1202 (AASHTO T 277) *Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration.* This test, discussed previously in this guideline, has excellent correlation to the results obtained by AASHTO T 259 for a wide variety of concrete types and qualities (except for concrete made with silica fume). The rapidity, ease of use, and reliability make this test very attractive when investigating the chloride ion permeability characteristics of concrete. Table III-13 can be used to make a general assessment of the chloride ion permeability of the concrete tested using this test method (ASTM C 1202). Because of its usefulness, it is recommended that this test be routinely conducted during mix design as a tool to assess concrete permeability, as long as its limitations are recognized, especially when certain admixtures are used.

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

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

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

NEED FOR RECORD KEEPING

This section of the guideline has included information regarding the selection of materials, mixture design, and construction practices to help prevent the occurrence of MRD in concrete pavements. Due to the complexity of the deterioration mechanisms, and to some degree the lack of accurate data from inservice concrete pavements, there is some uncertainty regarding the prevention of many MRD types. SHAs are therefore encouraged to maintain detailed records of newly constructed concrete pavements to assist in studying the effectiveness of various preventive measures. ACI 126.1R, *Guide to a Recommended Format for the Identification of Concrete in a Materials Property Database* (ACI 1997) provides a recommended format for use in computerization of concrete materials property data for entry into a database. This detailed guide provides the required forms and examples of how to complete them. SHAs are highly encouraged to collect such data, as the use of a standardized data collection format by all SHAs will provide a unique opportunity for researchers to examine MRD occurrences over a wide geographical area. This will assist in answering some of the unknowns that currently exist in the understanding of deterioration mechanisms, ultimately leading to improvements in concrete pavement performance.

SUMMARY OF TREATMENT, REHABILITATION, AND PREVENTION

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

APPENDIX A – FIELD DATA COLLECTION FORMS

Field Survey Equipment Checklist

Item	✓
Measuring Wheel	
Marking Paint	
Faultmeter	
Ruler/Scale	
Pocket Knife	
Clipboard/Pencils	
Water Spray Bottle	
Magnifying Glass	
Camera or Camcorder (with extra film/tapes)	
Flashing Warning Light	
Safety Vests and Hard Hats	
LTPP Distress Identification Manual	
Project Location	
Local Contact Name/Number	

GENERAL PROJECT INFORMATION FORM

Project ID: ______________

County: ________________

Project Identification

Climatic Zone: W-F D-F W-NF D-NF **Avg. # of F-T Cycles:** _____

Avg. Precip.: ______________ **Avg. Low Temp.:** ______________

Avg. Low Temp.: ___________ **Deicer Use:** Yes No

Highway No. ________________

Nearby Town: ________________

ADT: ________________

Percent Trucks: ________________

Type of Highway: Interstate U.S. State Other

Direction: NB SB EB WB

No. of Thru Lanes (1 Direction): ________________

Project/Section Location

	Milepost Start	Milepost End	Station Start	Station End	Direction	Cut/Fill (C or F)	C or F Amount	Depth of Ditch
Proj. Limits						▨	▨	▨
Sect. 1								
Sect. 2								
Sect. 3								
Sect. 4								

Project Design

Pavement Design

Pavement Type: ________________

Slab Thickness: ________________

Skewed Joints: Y N if yes, skew: ________

Joint Spacing: ________________

Load Transfer: ________________

Trans. Joint Sealant: ________________

Long. Joint Sealant: ________________

Long. Jt. Form Method: Sawed Insert Butt

Subgrade Classification: ________________

Construction Date: ________________

Base Type: ________________

Base Thickness: ________________

Subbase Type: ________________

Subbase Thickness: ________________

Shoulder Type: AC PCC Gravel Turf

Shoulder Width: Outside ____ Inside ____

Subdrainage Type: ________________

Design k-value: ________________

Comments: ____

____

____

Mix Design Information

Project ID: ______________ **Dates of Construction:** ____/____/____ to ____/____/____

Section: ______________

Cementitious Materials

Cement Type: ________________ **Source:** ________________________

Total Alkalis (%): ________________ (Na equiv.) **Fineness:** ________________________

Mill Certification?: Yes No If "Yes", Attach mill certification to this form.

Fly Ash Type: ________________ **Source:** ________________________

Total Alkalis (%): ________________ (Na equiv.) **Fineness:** ________________________

Certification?: Yes No ________________ If "Yes", Attach certification to this form.

Other Finely Divided Materials?: Yes No If "Yes", describe and attach documentation.

____

PCC Mixture Design

Design Weight Proportions (kg/cubic meter or pounds/cubic yard)

Cement: ________________ **Other Cementitious:** ________________________

Fly Ash: ________________ **Fine Aggregate:** ________________________

Water: ________________ **Coarse Aggregate:** ________________________

Unit Weight: ________________ **Percent Air:** ________________________

***w/c*:** ________________ **Compressive Strength:** ____________ @ ________ days

Slump: ________________ **Other Standard Tests:** ________________________

____

Admixtures

Air Entrainment

Source: ________________ Type: ________________ Rate: ________________

Admixture 2

Source: ________________ Type: ________________ Rate: ________________

Admixture 3

Source: ________________ Type: ________________ Rate: ________________

Comments: ____

____

Concrete Fine Aggregate

Project ID: ____________

Date of Construction: ____/_____/_____

Additional Identifiers:

____

____

Fine Aggregate No. 1

Source: ____________

O.D. Unit Weight: ____________

O.D. Specific Gravity: ____________

Absorption: ____________

Weight in Mix: ____________

Gradation (% Passing)

4.75 mm	2.36 mm	1.18 mm	600 µm	300 µm	150 µm	75 µm

Lithologic Type and Mineralogy:

____

____

____

FM: ____________

Clay (%): ____________

Chert(%): ____________

AAR Testing: ____________

Performance: ______________________________

____

Fine Aggregate No. 2

Source: ____________

O.D. Unit Weight: ____________

O.D. Specific Gravity: ____________

Absorption: ____________

Weight in Mix: ____________

Gradation (% Passing)

4.75 mm	2.36 mm	1.18 mm	600 µm	300 µm	150 µm	75 µm

Lithologic Type and Mineralogy:

____

____

____

FM: ____________

Clay (%): ____________

Chert(%): ____________

AAR Testing: ____________

Performance: ______________________________

____

Concrete Coarse Aggregate

Project ID: __________

Date of Construction: _____/_____/_____

Additional Identifiers:

____

____

Coarse Aggregate No. 1

Source: __________

O.D. Unit Weight: __________

O.D. Specific Gravity: __________

Absorption: __________

Weight in Mix: __________

Gradation (% Passing)

63 mm	50 mm	37.5 mm	25 mm	19 mm	12.5 mm	9.5 mm

4.75 mm	2.36 mm	1.18 mm	600 μm	300 μm	150 μm	75 μm

Lithologic Type and Mineralogy:

____

____

____

LA Abrasion: __________

Clay Lumps (%): __________

Chert(%): __________

Soundness: __________(Na or Mg)

F-T Durability: __________

AAR Testing: __________

Performance: ____________________

____

Coarse Aggregate No. 2

Source: __________

O.D. Unit Weight: __________

O.D. Specific Gravity: __________

Absorption: __________

Weight in Mix: __________

Gradation (% Passing)

63 mm	50 mm	37.5 mm	25 mm	19 mm	12.5 mm	9.5 mm

4.75 mm	2.36 mm	1.18 mm	600 μm	300 μm	150 μm	75 μm

Lithologic Type and Mineralogy:

____

____

____

LA Abrasion: __________

Clay Lumps (%): __________

Chert(%): __________

Soundness: __________

F-T Durability: __________

AAR Testing: __________

Performance: ____________________

____

Construction Information

Project ID: ______________ **Dates of Construction:** ____/____/____ to ____/____/____

Section: ______________

Mixing Plant

Type: ______________ **Make:** ______________________

Capability: ______________ **Mixing Cycle:** ______________________

Comments: ____

____

Construction

Type of Truck: **Mixing in Transit?:** Yes No

Haul Time: ______________ **Curing Method:** ______________________

Comments: ____

Mix Properties During Construction

Range and Avg. Slump:______________ Range and Avg. Air:______________

Range and Avg. Strength:______________ Type:______________

PCC Mixture Temperature at Placement

Average:______________ Date:______________

Low:______________ Date:______________

High:______________ Date:______________

______________ ______________

Comments: ____

Climatic Conditions During Paving

High Air Temperature:______________ Date:______________

Low Air Temperature:______________ Date:______________

Average Air Temperature:______________

Comments on Humidity: ____

Comments on Wind: ____

Does there appear to be a relationship between climatic conditions and observed distress? If so, prepare a plot of mixture temperatures along the project length to see if a correlation exists.

FIELD SURVEY DATA COLLECTION FORM

Date of Survey: ______/______/______

Survey Crew: ______ ______ ______

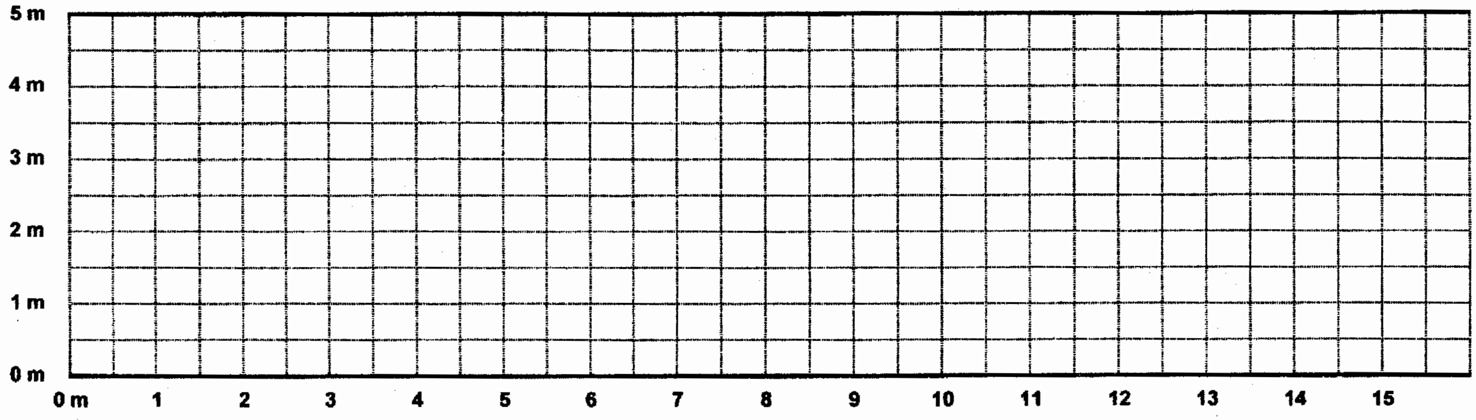
INSTRUCTIONS:

1. Sketch in longitudinal and transverse joints and note spacing only on first survey sheet.
2. Draw the following distresses on survey sheets: corner breaks, longitudinal cracking, transverse cracking, spalling, joint seal damage, general map cracking/scaling areas, polished aggregate, popouts, pumping, and patches.
3. Record joint faulting on this sheet below the joint.
4. Use *MRD Typical Slab Distresses* survey sheet to sketch individual materials distresses for a typical slab.

Project ID: ______________

Section Number ______

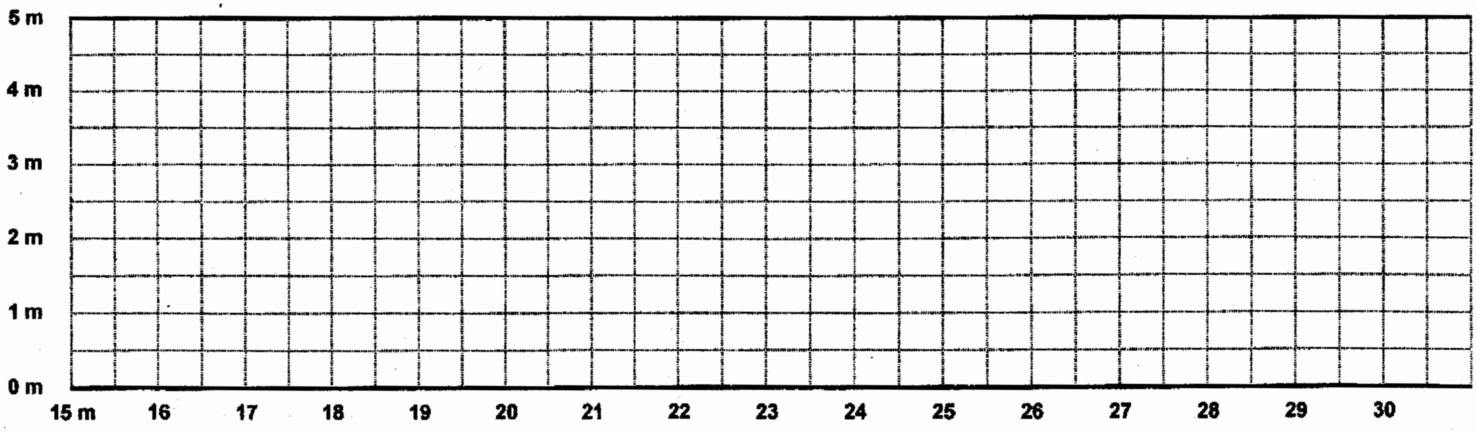
Page No. ____ **of** ____



Faulting 1 ft from slab edge ______

Faulting 2.5 ft from slab edge ______

North Arrow



MODIFIED LTPP DISTRESS SUMMARY FORM

Date of Survey: ______/______/______

Survey Crew: ______ ______ ______

Project ID: ______________

Section Number _______

Inspection Site _______

	Distress Type	Distress Measure	Severity Level			Comments
			Low	Moderate	High	
Cracking	Corner Breaks	number				
	Long. Cracking	meters				
		lin meters sealed				
	Trans. Cracking	number				
		lin meters				
		lin meters sealed				
Tr. Jts	Sealed	# of joints				
	Damaged Sealant	lin meters				
	Spalling	lin meters spalled				
Long. Jts	Sealed	# of joints				
	Damaged Sealant	lin meters				
	Spalling	lin meters				
Surf. Conditions	Map Cracking	# of slabs				
		sq. meters				
	Scaling	# of meters				
		sq. meters				
	Polished Aggregate	sq. meters				
	Popouts	#/sq. meter				
Other	Blowups	number				
	Trans. Joint Det.	number				
	Flexible Patches	number				
		sq. meters				
	Rigid Patches	number				
		sq. meters				
	Punchouts	number				
	Pumping/Bleeding	# of occurrences				
		lin meters affected				

TYPICAL SLAB MRD MANIFESTATIONS

Date of Survey: ______/______/______

Survey Crew: ______ ______ ______

Project ID: ______________

Section Number _______

Inspection Site _______

INSTRUCTIONS:
1. Select scale and sketch outline of typical slab(s) with MRD.
2. Draw typical crack pattern and location on slab(s).

Location of Slab(s) Within Survey Section: Sta: +

→

Traffic

MRD CHARACTERIZATION FORM

Date of Survey: ______/______/______

Survey Crew: ______ ______ ______

Project ID: ______________

Section Number _______

Inspection Site _______

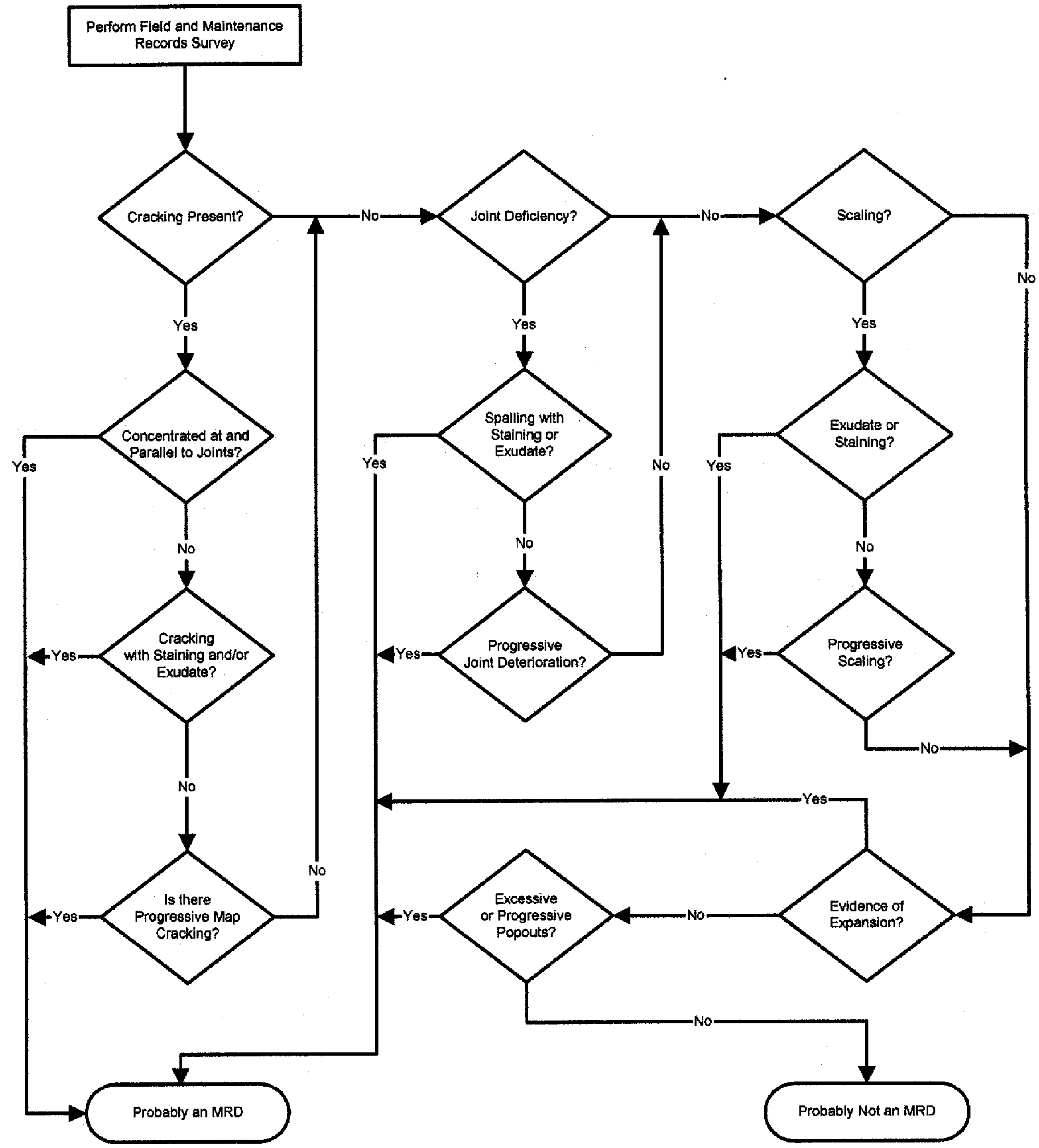
Cracking	Location	Joints	Cracks	Above Steel	Entire Slab	Other/Comments:
	Orientation/Shape	Parallel	Semi-circle	Criss Cross		Other/Comments:
	Extent	25% slab	50% slab	75% slab	Entire Slab	Other/Comments:
	Crack Size	Hairline	Open	Spalling		Other/Comments:

Staining	Location	Joints	Cracks	Above Steel	Entire Slab	Other/Comments:
	Color	Dark Gray	White	Rust		Other/Comments:

Exudate	Present	Yes	No			Other/Comments:
	Color	Dark Gray	White	Rust		Other/Comments:
	Extent	Low	Moderate	High		Other/Comments:

Scaling	Location	Cracks	Joints	Above Steel	Entire Slab	Other/Comments:
	Area of Surface	< 10%	10% to 25%	25% to 50%	> 50%	Other/Comments:
	Depth	< 3 mm	3 to 6 mm	6 to 12 mm	> 12 mm	Other/Comments:

Vibrator Trails	Visible Trails	Yes	No	Other/Comments:
	Discolored	Yes	No	Other/Comments:
	Distressed	Yes	No	Other/Comments:
	Change in Texture			Other/Comments:



Flowchart for Assessing Likelihood of MRD

LABORATORY LOG OF PCC PAVEMENT CORES

Sheet ____ of ____

Project Designation: ______________ State: ____________________ Highway: _____________
Nearby City and Distance: ____________________________________ Direction: _____________
No. of Through Lanes (in direction sampled): ____________________ Lane Sampled: _________
Beginning Milepost/Station: ____________________ End Milepost/Station: _________________

Operator: ___________________________ Core Diameter:_____________________________
Coring Date: _________________________ Core Barrel Tip Type:_______________________
Job ID:_____________________________

Note: Each column shown below should be used to record information for all cores/pieces extracted from a single panel. "Depth" should be measured from the pavement surface to the bottom of the core/piece and recorded to the nearest 2 mm. Front direction is the direction of traffic.

CORE NUMBER	A	B	C	D	E	OTHER
CORE ID						
LOCATION/STATION						
Offset From Joint						
Offset From Shoulder						
Number of Pieces?						
Core Piece No. 1						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 2						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 3						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
Core Piece No. 4						
Position *(Left, Right, Back, Front of core)*						
Depth (mm)						
REMARKS						

Core Identification Label

General Information

State:__________ **Highway:**____________________

Nearby City and Distance:____________________

Direction of Section Sampled:____________________

No. of Thru Lanes (in section direction):____________________

Lane Core Retrieved From:____________________

Begin Milepost/Station No. of Section:____________________

Core Identification Code

Project: ________________

Section: ________________

Inspection Site: ____________

Coring Slab: ____________

☐☐
Core ID

Core ID
A = joint core over dowel
B = joint core between dowels
C = joint core 0.3 m away
D = center core free of MRD
E = center core with MRD
O = other (describe below)

Sketch

Comments

Completed By: Date:

APPENDIX B: LABORATORY METHODS

This appendix has specific information about the laboratory techniques used to analyze concrete specimens. This information is provided for quick reference and the reader is directed to the literature recommended in this guideline for detailed descriptions of the test methods. The following laboratory methods are discussed:

B.1 Determination of Water-to-Cement Ratio (*w/c*)
B.2 Staining Concrete to Identify Sulfates and Alkali–Silica Reactivity (ASR)
B.3 Optical Microscopy (OM)
B.4 Scanning Electron Microscopy (SEM)
B.5 X-ray Diffraction (XRD)

B.1 DETERMINATION OF WATER-TO-CEMENT RATIO

Introduction

Determination of *w/c* by chemical methods involves independently determining the apparent water and apparent cement content, and then calculating *w/c*. The free water is determined by a combination of saturation/drying and weighing methods consistent with the methods described in ASTM C 642 *Standard Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete.* Determining the combined water requires drying the concrete at 550 °C and recording the weight loss.

Equipment

These procedures require the following major equipment or equivalent:

- Jaw crusher.
- Disk pulverizer or Roller Mill.
- Conventional drying oven.
- High temperature oven (550°C).
- Laboratory balance with sensitivity to 0.0001 gram.

Cement content can be determined using one of the methods described in ASTM C 1084 *Test Method for Portland Cement Content of Hardened-Cement Concrete*. One method accomplishes preferential dissolution of the cement in concrete using a cold-HCl solution. The other method presented, and the one adopted for this guideline, uses methanolic maleic acid (MMA) to digest the cement. This procedure requires less equipment then the cold HCl method. The major required equipment is as follows:

- Ventilated hood.
- 3 to 6 1000-ml beakers (depending on number of samples analyzed simultaneously).
- Stir plate and magnet for each beaker.
- Buchner filter and papers.
- Vacuum filter flask (2000 to 4000 ml).
- Conventional drying oven.
- Laboratory balance with sensitivity to 0.0001 gram.

Method

Clearly, accurate determination of the *w/c* value in a failed concrete section is an important part of understanding why the material failed. This is of course due to the known relationships between *w/c*, strength, and durability. Methods for determining *w/c* have been developed based on chemical laboratory methods. These methods involve independently determining the water and cement content and then calculating *w/c*. The free water is determined by a combination of saturation/drying and weighing methods consistent with the methods described in ASTM C 642 *Standard Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete*. Determining the combined water requires drying the concrete at 550 °C and recording the weight loss. Cement content can be determined using methods described in ASTM C 1084 *Test Method for Portland Cement Content of Hardened-Cement Concrete*. This commonly applied method accomplishes preferential dissolution of the cement in concrete using a cold-HCl solution. However, this solution is known to attack some aggregates resulting in erroneous cement contents (Hime 1993). The method presented in this report is adapted from the method presented by Marusin (Marusin 1981) using MMA to digest the cement. The procedure is presented below.

Step A: Determine Mixing Water (MW) content

1. Cut a 25-mm sample longitudinally from each core.
2. Saturate slab by submerging in water for 48 hours.
3. Weigh submerged (Wt_{WATER}) and then blot dry and immediately weigh in air (Wt_{SSD}).
4. Calculate Unit Weight – UW_{SSD} in $Kg/m^3 = (Wt_{SSD} / (Wt_{SSD} - Wt_{WATER})) \times 1000$.

Note: Special care and due speed is required for steps 5 and 6 to prevent loss of moisture to evaporation.

5. Crush slab in a jaw crusher to speed drying.
6. Immediately weigh sample (Wt_{WET}) and dry overnight at 105 °C. Re-weigh after drying to determine the dry weight (Wt_{DRY}).
7. Calculate Free Water $(FW_{SSD}) = ((Wt_{WET} - Wt_{DRY}) / Wt_{WET}) \times 100$.
8. Pulverize the sample to -50 mesh and blend. Exercise care to avoid loss of dust which is typically rich in cement.
9. Use riffle splitter to obtain a representative 1-gm sample. Dry at 550 °C for 4 hours and weigh to determine the oven dehydrated weight ($Wt_{550\,C}$).
10. Calculate Bound Water dry $(BW_{dry}) = ((Wt_{DRY} - Wt_{550C}) / Wt_{DRY})$.
11. Calculate Bound Water SSD $(BW_{SSD}) = BW_{dry} \times (1 - FW_{SSD})/100)$.
12. Determine aggregate absorption = percent aggregate (SSD) x percent absorption.
13. Calculate Mixing Water (MW) content corrected for aggregate absorption:

$$MW_{SSD} = (BW_{SSD} + FW_{SSD}) - \text{aggregate absorption}$$

Step B: Determine Cement Content

1. Dissolve 10 gm of sample in 70 gm of maleic acid and 400 ml of methanol for 30 minutes.
2. Allow sample to settle for 30 minutes.
3. Vacuum filter through #40 filter paper and wash thoroughly with methanol.
4. Dry at 105 °C to constant weight.
5. Cool in a desiccator and weigh residue.
6. Calculate Percent Cement (SSD) = $(100 - \text{residue} - BW_{dry}) \times (1 - FW_{SSD})/100$).
7. Calculate Cement Content (SSD) in Kg/m^3 = Percent Cement (SSD) x UW_{SSD}.

Step C: Determine Water-to-Cement Ratio

1. Calculate $w/c = MW_{SSD}$ / Cement Content (SSD).

B.2 STAINING CONCRETE TO IDENTIFY SULFATES AND ASR

Introduction

Generally speaking, staining can be carried out in any lab space and may very well be conducted in the petrographic lab if no chemistry lab space is available. The one exception to this is the SHRP uranyl acetate test (Stark 1991). The primary concern with conventional staining is storage of the chemicals. A closed, ventilated, dry, and secure cabinet for storing chemicals is required. Also, a waste stain collection bottle should be used for each type of stain so the unused stain can be reused without re-mixing spent and fresh stain.

It is noted that all of the stains used are hazardous materials and it is therefore necessary that strict policies be developed and followed for handling and disposal. All stains should be clearly labeled and dated.

Equipment

The required equipment for staining depends on the method being used. Excepting the SHRP uranyl acetate test, standard laboratory beakers, Nalgene storage bottles, nitrile gloves, and safety goggles are all that are required. For staining large sections, a medium depth plastic pan (dishpan) is useful. The SHRP uranyl acetate test requires a special lighting fixture and protective glasses for viewing the stained regions. These lighting devices are available as part of a SHRP testing kit from a variety of vendors.

Staining Techniques

Staining methods can be conducted in the field but are best carried out in a laboratory where safe and proper storage and disposal of chemicals is accommodated. Four existing staining methods are presented. These include the barium chloride potassium permanganate (BCPP) stain for sulfate minerals in concrete (Poole and Thomas 1975), the recently introduced sodium cobaltinitrite/rhodamine B method for ASR (Guthrie and Carey 1997), the SHRP uranyl acetate method for identifying ASR in concrete (Stark 1991), and phenolphthalein to determine the depth and extent of paste carbonation.

Barium Chloride Potassium Permanganate (BCPP) Stain for Sulfate Minerals

The BCPP stain is very useful for identifying ettringite, gypsum, and anhydrite phases in concrete. The principal staining method is a two-step process. The concrete is initially immersed in the stain solution, causing sulfate ions released from the concrete to precipitate as barium sulfate. As this occurs, the potassium permanganate co-precipitates with the barium sulfate imparting the characteristic purple color to the resulting crystal. This process results in a permanent alteration of the sulfate phase surface at the sulfate/water phase boundary. The concrete is then rinsed in a saturated solution of oxalic acid to remove any surface coloration from precipitation of excess potassium permanganate. The remaining purple colored crystals identify where ettringite, gypsum, or anhydrite phases existed.

Step A: Prepare Solutions

1. Prepare a solution of barium chloride, potassium permanganate, and water where barium chloride and potassium permanganate are mixed in a 2:1 ratio and the total amount of solute is 6 percent of the solution by weight.
2. Prepare a saturated solution of oxalic acid.

Step B: Stain and Rinse

1. Remove dust and rock flour residue from concrete slab with compressed air.
2. Rinse surface to be stained with distilled water.
3. Briefly dry surface to be stained until surface approximates a surface saturated dry (SSD) condition.
4. In the plastic tub, apply barium chloride – potassium permanganate solution to surface, allow to sit for 2 minutes.
5. Briefly rinse surface with distilled water, making sure to collect drippings in the plastic tub.
6. Empty and rinse the plastic tub.
7. Briefly rinse surface with oxalic acid solution, making sure to collect drippings in the plastic tub.
8. Pat surface dry with paper towel, taking care to remove any reservoirs of oxalic acid that may sit in the large entrapped air voids.
9. Empty and rinse the plastic tub.
10. Properly dispose of soiled paper towels and gloves.

Sodium Cobaltinitrite/Rhodamine B Staining Method for ASR Reaction Products

The sodium cobaltinitrite/rhodamine B method is relatively new and as a result, little field experience currently exists regarding its applicability and reliability. Yet some aspects of the technique appear to be very promising, addressing some limitations noted in the uranyl acetate method. The recommended test sequence is as follows:

1. Remove dust and rock flour residue from concrete slab (gently blast with air hose).
2. Rinse surface to be stained with distilled water.
3. Briefly dry surface to be stained until surface approximates a surface saturated dry (SSD) condition.

4. In the plastic tub, apply sodium cobaltinitrite solution to surface, allow to sit for 30 to 60 seconds.
5. Briefly rinse surface with distilled water, making sure to collect drippings in the plastic tub.
6. Pat surface dry with clean paper towel until it approximates a surface saturated dry (SSD) condition.
7. In the plastic tub, apply rhodamine-B solution to surface, allow to sit for 30 to 60 seconds.
8. Briefly rinse surface with distilled water, making sure to collect drippings in the plastic tub.
9. Empty and rinse the plastic tub.
10. Pat surface dry with clean paper towel.
11. Properly dispose of soiled paper towels and gloves.

After drying, the specimens can be visually assessed with a hand lens or using an optical microscope. Regions affected by ASR stain either yellow or pink. It is observed that yellow staining is associated with a massive precipitate having a distinctive gel-like morphology and granular precipitate that appears to consist of crystals that have grown from the gel (Guthrie and Carey 1997). It has been determined that the yellow stained regions were K+Ca+Si±Na reaction products resulting from ASR.

The literature states that pink staining regions were found to be generally less prevalent and isolated in highly deteriorated areas and that they commonly occurred only when extensive yellow staining also existed in the sample. Interestingly, yellow staining was often present in the reaction rims around aggregate while the pink staining was outside this area along fractured faces. Prior to staining, the entire reaction product appeared white, but after staining, it had clearly defined areas of yellow and pink. The pink staining material was identified as being alkali poor Ca+Si gels associated in some way with advanced ASR (Guthrie and Carey 1997). It is hypothesized that the pink staining gels formed from yellow staining gels either through leaching of alkalis or through a reaction with the cement paste. In practice, it has been observed that the rhodamine-B stained the calcium rich hydrated cement paste pink, therefore limiting the usefulness of this stain. As a result, it might be desirable to use the sodium cobaltinitrite alone as an indicator for ASR.

Guthrie and Carey (1997) compared the results of their staining method to the uranyl acetate method. They found excellent agreement between the two methods with the added benefit that their dual staining technique provided additional information through the appearance of both yellow and pink staining (the uranyl acetate test tagged both). The major benefit of cobaltinitrite/rhodamin B method is that the chemicals are not as rigorously regulated as uranyl acetate and that a UV light source is not required for illumination. Thus conventional optical microscopes (OM) can be used for viewing stained specimens.

SHRP Uranyl Acetate Test for ASR Reaction Products

In the SHRP uranyl acetate test, concrete is treated with a uranyl acetate solution, which is then rinsed off. This leaves uranyl ($UO^{2+}{}_2$) sorbed to the negatively charge ASR gel which fluoresces with yellow-green glow under UV light (Natesaiyer and Hover 1992). It is this glow that is attributed to the presence of ASR gel.

Although this test has been very helpful to the practitioner, it has a number of problems associated with it. One major problem is the key ingredient of the stain is radioactive uranyl acetate. Although the concentration of uranyl acetate is relatively small, it is significant enough to warrant special handling and disposal procedures from a regulatory standpoint. Therefore, obtaining permission to receive this material at some labs is not trivial and documentation is often required. Consult with your radiation safety personnel or State agencies governing radioactive materials as to specific laws in your State. Once received, this material must be stored in a closed, ventilated, dry, and very secure cabinet to minimize any chance of spillage. The radiation given off is minimal and radiation shielding is not required in the storage cabinets. Once exposed to uranyl acetate, the concrete becomes contaminated and must be disposed of using acceptable procedures.

Another disadvantage of the uranyl acetate test is that it requires the use of a UV light source to fluoresce the gel. This is problematic as it makes it almost impossible to simultaneously view the gel and other relevant concrete characteristics such as void structure, aggregate, paste, and fractures. This is also a limiting factor in field analysis where bright sunlight may make it difficult for the analyst's eyes to adjust for adequate viewing of the concrete under UV light.

The final disadvantage is that the uranyl ion is nonspecific, meaning that it will associate with most cations in concrete (Guthrie and Carey 1997). This nonspecificity means that false positives are not uncommon when using the uranyl acetate test, possibly leading one to believe that there is an ASR problem when it actually might not exist.

Phenolphthalein Staining Method for Determining Depth and Extent of Carbonation

The use of phenolphthalein as a stain for determining the depth and extent of carbonation is based upon the pH of the cement paste. Any cement paste within the concrete that has a pH greater than 10 will be stained a magenta color. All other constituents including carbonated paste remain unchanged. Carbonation results in reduction of pH in the paste and thus carbonated paste will not stain. The required equipment is the same as the BCCP stain, but instead a 0.5 percent phenolphthalein in methanol solution is used. The procedure is as follows:

1. Remove dust and rock flour residue from concrete slab (blast with air hose).
2. Rinse surface to be stained with distilled water.
3. Briefly dry surface to be stained until surface approximates a surface saturated dry (SSD) condition.
4. In the plastic tub, apply alcoholic phenolphthalein solution to surface, allow to sit for 10 seconds.
5. Pat surface dry with clean paper towel.
6. Empty and rinse the plastic tub.
7. Properly dispose of soiled paper towels and gloves.

The distinct difference in color allows for directly imaging the depth of carbonation using a stereo OM or, if it is a polished slab, a computer scanner.

B.3 OPTICAL MICROSCOPY

Introduction

The most common types of concrete specimens observed in an optical microscope are broken pieces, polished slabs, picked grains, and thin sections. Broken pieces require no sample preparation if examined by stereo OM. This is often very useful for examining fractures, aggregates, and filled voids. Also, reaction products can sometimes be detected and sampled, or picked, for analysis by petrographic OM or scanning electron microscope (SEM). A polished slab is commonly more useful than a broken piece because the cross section of a whole core can be examined. All constituents of the concrete can be seen and examined in detail. Often, staining is applied to the polished slab to reveal specific types of features. Aggregate type and abundance can be readily determined from a polished slab and cracks are easily seen and traced through the concrete. Reaction rims and products are clearly observed in polished slabs. Picked grains are minute pieces of concrete extracted from a specimen, usually while observing it in the stereo microscope. These grains may be analyzed on the petrographic microscope using refractive index liquids or on the SEM. Thins sections are 20-micron-thick pieces of concrete bonded to a glass slide commonly using fluorescent dyed epoxy. These are useful for transmitted light observation on the petrographic OM and backscattered imaging in the SEM.

Equipment

Sample Preparation Equipment

The major pieces of equipment required are as follows:

- 610-mm diamond sectioning saw (for cutting slabs from a core).
- 150-mm diamond sectioning saw (for cutting small pieces).
- 405- to 510-mm vibrating lap (depending upon slab size).
- 250- to 460-mm grinding wheel (depending upon slab size).
- 250- to 460-mm polishing wheel (depending upon slab size).

Thin sections are often required when examining concrete to allow for identification of components in the concrete microstructure. These specimens require practice to prepare and require more elaborate equipment as compared to polished slabs. The required major equipment is as follows:

- 610-mm diamond sectioning saw (for cutting slabs from a core).
- 150-mm diamond sectioning saw.
- 150-mm diamond wafering saw (optional, for cutting section thin before grinding).
- Thin section grinder or 460-mm lap (with 600 grit silicon carbide).
- 250- to 460-mm polishing wheel or glass plate with silicon carbide.

Usually, thin sections are epoxy impregnated to fill voids and cracks, improving the image quality in the OM. In the case of fluorescence microscopy, the epoxy contains a fluorescent dye that must be mixed in the epoxy prior to use. Detailed instructions for impregnating specimens and preparing the fluorescent dye are given in various manuals (Roy et al. 1993a; Walker 1992). The additional equipment needed to perform epoxy impregnation includes:

- A vacuum chamber (commercially available with *in situ* epoxy delivery system).
- A vacuum pump.

The Stereo Zoom Optical Microscope

The stereo zoom OM is arguably the most useful tool for examining distressed concrete. A typical stereo zoom OM can provide magnifications to 150 to 200x and has a considerable depth of field at any given magnification allowing for observations of cracks, voids, and crystalline grains. It is recommended that all laboratories analyzing concrete obtain a quality stereo zoom OM. When purchasing one, some desirable features are click stops on the magnification control, multiple types of light sources (e.g., gooseneck, ring lights), a sturdy, vibration isolated adjustable boom stand, and a maximum magnification of at least 140x. An automated stage may be useful for manipulating the specimen. Most commercially available stages provide programmability so, if in-house support is available, software can be written to perform air-void system analysis in accordance with ASTM C 457. Digital frame grabbers are also available to capture digital images from the microscope.

The Petrographic Microscope

The petrographic OM is also very useful but requires considerably more acquired skill as compared to the stereo OM. The petrographic OM can be used to identify various constituents in concrete by passing plane polarized illumination through a prepared thin section. As light passes through materials, it bends, or refracts, with the degree of refraction dependent upon the material. Light traveling through most translucent crystalline materials is broken down into two rays that behave differently, with one being decelerated (extraordinary ray) and one unaffected (ordinary ray). The refraction of these waves within a crystal varies as the specimen is rotated and this refraction results in image contrast that can be used to identify different mineral species. When purchasing a petrographic OM, the key features are:

- Rotating stage.
- Substage condenser w/apertures.
- Polarizing filter.
- Polarizing analyzer.
- Bertrand lens.
- Gypsum accessory plate.
- Refractive index liquids.
- Quality lenses.

The choice of lenses is very important and should be thoroughly considered. The lenses are the heart of the microscope and will determine the quality of the image. Some common lens types are achromats, apochromats, and fluorites. The term "plan" is included in the lens type if it has been corrected to remove the curvature of the lens from the resulting image. Generally, buy the best lens you can afford. If fluorescence microscopy is to be used, the highest quality lenses will almost certainly be required. The two common descriptors of a lens are the magnification factor and numerical aperture (NA). Common magnification factors are 3.5x, 5x, 10x, 20x, and 50x. The numerical aperture is a measure of the lens' resolution capabilities with an increased NA indicating a higher resolution lens.

Refractive index liquids are used to identify materials mounted as grains on a microscope slide. As a beginning set, the following refractive index liquids are recommended:

- Refractive index of 1.460-1.640 in increments of 0.004.
- Refractive index of 1.642-1.700 in increments of 0.01.
- Refractive index of 1.705-1.800 in increments of 0.005.

As with the stereo OM, some form of image recording is required. Petrographic OMs typically support 35-mm cameras or video camera/frame grabber systems.

Automated Optical Microscopes

To perform an ASTM C 457 analysis, some type of automated microscopy system is preferred. A number of small firms offer limited production turnkey systems that perform this test in either a fully automated or semi-automated mode. Any system to be purchased should be compared to the systems described in ASTM C 457 to confirm compliance. In general, a semi-automated approach is almost always desired to allow the operator to make judgments about features in the microscope image. Fully automated systems have been developed to analyze specially prepared samples where the entire matrix has been blackened and the available pores filled with a white powder (e.g., wollastonite dust.) The resulting image is a "binary image" where only two colors exist and the computer can easily make judgments about what is being considered a void. One drawback to this approach is that no information about the degree of void infilling is obtained. If possible, try to purchase a system that will allow for determination of the percentage of filled air voids.

Methods for Preparing Specimens

The first step in microscopic evaluation of concrete is sample preparation. Detailed methods for preparing samples are given in a variety of references (Walker 1992; Roy et al. 1993a; ASTM C 457; ASTM C 856). Not all methods or all types of samples need to be analyzed in every case. It is up to the analyst to decide which test will provide insight into the mode of failure. The purpose of this section is to provide an overview of the common types of laboratory procedures used in preparing specimens to investigate MRD in concrete pavements.

Broken Pieces

Often, for a case of MRD in a highway structure, broken pieces are readily available in addition to cored specimens. Broken pieces can be very useful for identifying secondary deposits in addition to studying the nature of the fractured surfaces. When the surfaces are freshly broken, fresh gel may be seen in cases of advanced ASR. No special sample preparation is required for viewing broken pieces in a stereo OM. To view these pieces in a CSEM, a conductive layer must be deposited as described in section B.4 of this appendix.

Grain Mounts

Often, material can be picked from the surface of concrete using a metal pick tool. This material can be placed on a glass slide in a drop of liquid of known refractive index. The grains of material can then be viewed on the petrographic microscope to determine the material's optical properties such as refractive index and birefringence, from which the material can be identified. This process is usually done in conjunction with stereo OM observation. Table 6 in ASTM C 856 lists the important optical properties of common concrete phases.

Polished Slabs

A very useful way to examine concrete cores is to section a slab from the core and polish one or both surfaces for examination. This provides a large area for examination and also allows for examining concrete in longitudinal and transverse directions. The number of slabs required will be determined by the tests being run. For example, within ASTM C 457 there are provisions for the minimal surface area to be analyzed. To facilitate microscopic examination, the polished slab needs to be as flat as possible with opposite sides being parallel. Stains can be applied directly to the slab, as described in Section B.2, to identify phases in the concrete. Aggregate rock type and distribution can be readily determined for both the coarse and fine aggregate using a polished slab. If the core being sectioned is fragile, it may be necessary to stabilize the concrete with epoxy, Carnuba wax, or 1:5 solution of nylon fingernail hardener in acetone or methanol. The following method is used for preparing concrete slabs for microscopic inspection:

1. Cut a slice (approximately 25 mm thickness) from the core using the large sectioning saw.
2. If required, further section the slab with the 150-mm sectioning saw to obtain a slab small enough for the grinding and polishing wheels used.
3. Further stabilize the slab, if necessary, with epoxy or other material.
4. Grind the slice with 200- through 400-grit silicon carbide on the 460-mm lap.
5. Polish with 600- through 1200-grit silicon carbide on a polishing lap on the 460-mm lap.

Epoxy Impregnation

Epoxy impregnation is often used to stabilize concrete, greatly facilitating most sample preparation techniques. In general, this should be done as a last resort because the epoxy impregnation is irreversible. Carnuba wax or 1:5 solution of nylon fingernail hardener in acetone or methanol are substances that can be dissolved away from the concrete, once hardened, making them a preferred stabilization method.

Often, impregnation is done for other reasons. It can be used to fill air voids with a material of known index of refraction or a material containing a fluorescent dye for fluorescent microscopy techniques. It is therefore common to use epoxy for this task. The following method is recommended for vacuum impregnating concrete with epoxy:

1. Perform all sectioning or grinding required before impregnation.
2. Place the surface to be impregnated face down in tray at the bottom of the evacuation chamber.
3. Prepare the epoxy mixture to be used, consulting the epoxy manufacturer's instructions. Usually a special epoxy with a very low viscosity is preferable. Consult manufacturer's specifications on any fluorescent or colored dyes added to the epoxy. Generally, the fluorescent dye, such as Fluorol Yellow 088 (produced by BASF Wyandotte Corp., Holland, Michigan), is initially added to the resin, and stirred overnight, making sure that it is protected from light. The exact dosage of dye varies but typical values are on the order of 1 percent by weight.

4. Place the epoxy mixture in a cup suspended above the tray in the evacuation chamber and start the vacuum pump.
5. Pump until a steady-state vacuum is obtained, making certain that the time it takes to achieve a steady-state vacuum does not exceed the pot-life of the epoxy mixture.
6. Tip the cup containing the epoxy so that it pours into the tray with the specimens.
7. Carefully bring the evacuation chamber back to atmospheric pressure. Introducing atmospheric pressure too quickly may cause the epoxy to splatter.
8. Allow the specimen to rest in the epoxy-filled tray for a period of 5 minutes. At this point, the specimen may be allowed to either remain in the epoxy-filled tray until the epoxy cures, or removed from the tray to wipe away excess epoxy. In the case of specimens to be used for thin-section production, it is advisable to press the impregnated surface against a flat sheet of plastic in order to squeeze away as much of the excess epoxy as possible. After the epoxy has cured, the specimen can be peeled from the plastic sheet.
9. Always ensure that the epoxy is protected from humidity throughout the curing process. This can easily be accomplished by enclosing the epoxy-impregnated specimen in a plastic bag.

Thin Sections

Thin sections are not difficult to produce but they are difficult to produce consistently well. There are a number of publications that detail the steps (Walker 1992, Roy 1993) and only a brief summary is given here. The art lies in the final polishing required to bring the section down to the desired thickness. Unfortunately, polishing is irreversible, and a specimen polished unevenly or too far is ruined. All of the sectioning and grinding steps are very important as the final thin section quality is greatly controlled by the uniformity and thickness of the slide before final polishing begins.

Thin sections are usually prepared on standard geological thin section slides using standard geological sample preparation methods. An ultimate thickness of 25 to 30 microns is obtained by first cutting and grinding the section to very near the desired thickness. Then, the section is thinned by hand until the desired thickness and taper is achieved. The latter is important as some petrographers purposefully taper their sections to provide a variety of thickness for observation. For example, thin sections near the standard thickness can be used for routine observations, while thinner areas are used for examining features such as unhydrated cement grains, or dolomite grains in ACR reactive aggregates. The following method is recommended for the preparation of thin sections:

1. Section small pieces of concrete into billets that can be bonded to a glass slide (27 by 46 mm). They should be fashioned to cover most of the slide and should be at least 10 mm in thickness.
2. Grind the surface of the billet that is to be bonded with the slide using 400-grit SiC, a lubricant, and a glass plate until the surface is smooth and flat. The choice of lubricant depends on the intended use of the thin section. If the thin section will be used for SEM analysis, it is advisable to avoid the use of water. Some alternatives to water include mineral oil, ethanol, or kerosene.
3. Rinse the surface of the billet, and follow with a sonic bath. Again, the choice of a cleaning medium depends on the intended use of the thin section. Some alternatives to water include ethanol or kerosene.

4. Repeat steps 2 and 3 using 600-grit SiC. Make certain that the cleaning medium is removed from the billet before moving on to the next step. This may generally be accomplished by leaving the billet in a 40 °C oven overnight, followed by a quick wipe with acetone and a paper towel.
5. Impregnate the prepared face of the billet as previously described in this guideline.
6. Further grind the impregnated surface using 1000-grit SiC, a lubricant, and a glass plate so as to just remove epoxy from the surface.
7. Rinse the surface of the billet, and follow with a sonic bath. Again, make certain that the cleaning medium is removed from the billet before moving on to the next step.
8. Bond the impregnated side of the billet to the slide with mounting epoxy. Always ensure that the epoxy is protected from humidity throughout the curing process. This can easily be accomplished by enclosing the specimen in a plastic bag.
9. Cut the billet from the glass slide, leaving at least a 0.2-mm-thick layer of the specimen bonded to the slide.
10. It may be desirable to repeat the impregnation procedure at this point, especially if high viscosity epoxy is used.
11. Grind the thin section to approximately 100 microns thickness. A thin section grinder is useful for this as it allows precise adjustment of the grinder head allowing for flat, uniform sections. Otherwise, the grinding may be performed by hand with 240-grit SiC, a lubricant, and a glass plate or lapping wheel.
12. Grind the thin section with 400-grit SiC, a lubricant, and a glass paste until a thickness of approximately 60 microns is achieved.
13. Rinse the thin section.
14. Grind the thin section with 600-grit SiC, a lubricant, and a glass paste until a thickness of approximately 40 microns is achieved.
15. Rinse the thin section.
16. Grind the thin section with 1000-grit SiC, a lubricant, and a glass paste until a thickness of approximately 30 microns is achieved.
17. Rinse the thin section.
18. Polish the thin section using 6-micron diamond paste and a polishing wheel covered with nylon cloth.
19. Rinse the thin section.
20. Polish the thin section using 1-micron diamond paste, a lubricant, and a polishing wheel covered with nylon cloth.
21. Rinse the thin section. Make certain that the cleaning medium is removed from the billet before moving on to the next step.
22. If the thin section is to be carbon coated for SEM, and either kerosene or mineral oils were used as a grinding or polishing medium, the thin section should be placed in a sonic bath with ethanol.
23. If the thin section is not intended for SEM examination, attach cover slip if desired with Lakeside 70 or similar adhesive.

B.4 ELECTRON MICROSCOPY

Introduction

The same specimens studied by optical microscopy can be studied using the scanning electron microscope (SEM). As a convention within these guidelines, the acronym SEM will refer to the general instrument including the high-vacuum conventional scanning electron microscope (CSEM), the environmental SEM (ESEM) and the low-vacuum environmental scanning electron microscope (LVSEM). Common types of concrete specimens for electron microscope observation are broken pieces, polished slabs, and thin sections. All specimens require a conductive coating if examined in the CSEM or in "CSEM mode" using an LVSEM or ESEM. Usually no coating is required for examination at low vacuum by the ESEM or LVSEM.

Equipment for the Electron Microscope

Scanning Electron Microscope (SEM)

The SEM is a convenient tool for identifying reaction products and other phases in distressed concrete. The SEM is also very useful for examining cracks, aggregates, and deposits in fractures or filled voids. The SEM forms an image by rastering a focused electron beam across a small rectangular area of a specimen. The electron beam and specimen interact resulting in two electron signals of interest and characteristic x-rays, to be discussed later. As the beam strikes the specimen, electrons from the material are released. These electrons are called secondary electrons, have a very low kinetic energy (<500 eV), and are used to form topographical images of a specimen. Electrons that originate in the electron beam have a very high energy (15-25 keV) and these electrons scatter in proportion to the specimen density at the point of impact. These backscattered electrons are used to form images showing specimen composition. As the electron beam is scanning the specimen area, the electron yield from the specimen is monitored using an electron detector. This electron signal is used to modulate the brightness or intensity of another electron beam scanning inside a viewing cathode ray tube (CRT). This modulation/scanning process maps out an image on the viewing CRT that is an "electronic representation" of what the specimen surface looks like. By design, an SEM produces images with extraordinary depth of field allowing for examination of rough surfaces.

When a point on a specimen is excited with the incident electron beam, characteristic x-rays are emitted from the specimen that are unique identifiers of the elements present in the irradiated volume. By identifying the energy or wavelength of the characteristic x-rays produced using a spectrometer, the elements present are identified. For a given incident beam current, the rate of characteristic x-ray production for a given element will be proportional to that element's concentration. This effect forms the basis of x-ray mapping and quantitative x-ray microanalysis.

Conventional Scanning Electron Microscope (CSEM)

In a conventional SEM (CSEM), the electron beam/specimen interaction occurs in a high vacuum (10^{-6} torr) allowing for transmission of the incident electron beam and emission of the low energy secondary electrons from the sample with a minimum of collisions between electrons and gas molecules in the chamber (i.e., high vacuum, low number of gas molecules in the chamber). This allows for the optimum operating conditions with a well-defined incident electron beam and a maximum yield of electrons from the specimen. However, this high vacuum creates severe dehydration and cracking of concrete, affecting the ability to make direct observations on cracking. In essence, the CSEM produces an image of altered concrete as compared to what was sampled. In many cases, this does not matter. ASR reaction products and secondary deposits can easily be identified based on their cation ratios. Other constituents, such as aggregates, are unaltered by the vacuum.

Environmental Scanning Electron Microscope (ESEM)

In an environmental SEM (ESEM), the electron beam/specimen interaction occurs at elevated pressures (0.2 to 20 torr), which greatly reduces or eliminates dehydration (4.6 torr is the minimum pressure that can sustain liquid water). With the pressure in the chamber set greater than the vapor pressure of water, dehydration and the associated cracking are reduced to near zero. Another benefit of the elevated gas pressure is it allows for discharging of any surface charge by ionization of the gas molecules. This reduces the need for conductive coatings. In combination, this greatly improves the applicability of electron microscopy to concrete imaging.

In the ESEM, collisions between incident beam electrons, emitted sample electrons, and gas molecules in the chamber are common. These interactions have both positive and negative effects. For the emitted secondary electrons from the specimen, the interaction with gas molecules results in ionization of the gas and a "gas amplification effect" that greatly enhances the electron signal strength. Special detectors have been developed to take advantage of this effect and this is one of the fundamental differences between an ESEM and an LVSEM. In an LVSEM, the backscattered electron signal is used for both compositional and topographical imaging. On the negative side, the high pressure environment creates a scattering and subsequent defocusing of the incident electron beam, leading to an uncertainty as to where the electron beam is actually striking the specimen. In imaging, this effect is not a problem as the randomly scattered electrons simply form a background to the electron signal that is easily filtered. In the case of x-ray microanalysis, it is a serious problem as x-rays could be coming from any part of the specimen and an element in the analyzed spectrum may in fact not be present at the assumed analysis point. In the end, the ESEM provides an excellent tool for imaging concrete and a limited tool for performing x-ray microanalysis when performed in the ESEM mode. An ESEM is capable of running at high vacuum for performing microanalysis.

Low Vacuum Scanning Electron Microscope (LVSEM)

The third type of scanning electron microscope is the low vacuum SEM (LVSEM). This instrument is very similar to a CSEM but has been adapted to also operate at elevated pressures (0.2 to 2 torr). In this environment, liquid water cannot be sustained and free water that is present freezes instantly at 4.6 torr and sublimates at lower pressures. For concrete thin sections or polished sections, there will be very little to no free water present if the specimens are not prepared using water. The water of hydration is chemically bound and will dissipate very slowly in the LVSEM. Although desiccation cracking will ultimately occur, it will be very slow process and will not be seen during a typical analysis. The gas environment of the LVSEM will also eliminate surface charging, eliminating the need for coating.

Purchasing a Scanning Electron Microscope

First, in comparing the different instruments there are obvious differences. Any of the three SEM types described can be used for analyzing concrete. A CSEM will provide the highest possible resolution and the best microanalysis data, but there will be cracking and dehydration, and the need to conductively coat specimens to eliminate surface charging. An ESEM is capable of functioning as an LVSEM or as a CSEM. However, design restrictions required for ESEM operation make the high vacuum performance less than optimum. An LVSEM operates well as a CSEM and allows for the examination of concrete with little to no dehydration cracking but liquid water cannot be sustained.

When deciding to buy an SEM, look for instruments with the capabilities for the largest possible sample size and stage travel. Many current SEM designs allow for samples with a diameter of over 200 mm, making possible observation of polished cross sections of cores. If the SEM is to be used for quantitative x-ray microanalysis, examine the geometry between the x-ray detector port and the specimen surface plane. The key parameter is the detector take-off angle, which is the angle of inclination between the specimen surface plane and the x-ray detector. A high take-off angle is preferred with 35 to 40° being the optimum. Other geometries may suffice and it is best to evaluate each instrument analytically with selected mineral standards before making a final decision. Also inspect the take-off angle of all other detectors. Avoid instruments with a negative take-off angle for any detector. That type of detector take-off angle puts the detector "behind" the specimen, which usually means a decrease in electron gathering efficiency.

Finally, when purchasing an SEM, evaluate the vendor and its performance by calling customers on its user list. Most instrument vendors provide this list upon request. Most importantly, evaluate the field service cost and performance. An SEM purchase is a major investment over the equipment's 10 to 20 years of service. In addition to the purchase price, expenditures for planned maintenance and supplies will be incurred and, thus, financial plans for addressing those costs before they occur need to be developed. As a rule of thumb, the maintenance cost of an SEM varies between 1 and 5 percent of the original purchase price, per year. This does not include the cost of an operator or analyst. For example, a first estimate of the support cost for a $300,000 SEM/EDS system is between $3000 and $15,000. The lower number represents a situation where all maintenance is conducted by the instrument operator and the high end represents a service contract purchased from the vendor.

X-ray Microanalysis

To perform x-ray mapping or microanalysis with an SEM, an x-ray analysis system must be purchased in addition to the SEM. The most common x-ray analysis system used on an SEM utilizes the energy dispersive (ED) spectrometer. The ED spectrometer, or ED detector, is an electronic device that determines the energy of individual x-ray photons. As previously stated, when a point on a specimen is excited with the incident electron beam, x-rays are emitted from the specimen that are characteristic of the elements present in the irradiated volume. By identifying the energy or wavelength of the characteristic x-rays produced using a spectrometer, the elements present are identified.

In an ED system, the spectrum of x-rays emitted by the specimen is established by assigning x-rays of similar energy to classification bins. For example, a 1eV x-ray would be in the 0-10eV bin and classified as 0ev. Likewise, a 11eV x-ray would be in the 10-20eV bin and classified as 10ev, etc. This bin process creates an X-Y histogram with bin energy on the abscissa and frequency of occurrence, or counts, on the ordinate. Characteristic x-rays appear as distinct distributions, or lines, superimposed on a slowly varying background. Usually, the identity of an element is determined by matching the families of lines occurring to preset cursor positions. This process is usually accomplished graphically. Once an element is identified, the integrated intensity of any one of its principal lines can be used as a measure of its concentration. The integrated intensity is extracted by defining a region called a window. This window is usually centered on the peak being integrated and adjacent bins either side of the peak centroid bin are summed, forming the integrated intensity for that element.

X-ray mapping is a way of forming an image that is based on characteristic x-ray information and therefore elemental information. In x-ray mapping, as in electron imaging, the incident electron beam is stepped or scanned over an area of the specimen. However, rather than modulate the viewing CRT electron beam with the specimen electron signal, the characteristic x-ray rate signal from pre-selected elements is used. For each element mapped, a window is defined in the ED spectrum. Each element is assigned a color and a separate map is developed for each element analyzed. These maps can be processed later to indicate common assemblages of elements. The intensity of any pixel in an x-ray map represents the relative intensity for that element, and therefore its relative concentration. The term relative indicates that all of the intensities measured in a digital x-ray map are typically normalized to the most intense pixel in the map. Therefore, direct comparison of intensities between different maps can only be done with prior knowledge of the conditions of analysis or by knowing the minimum and maximum raw intensities measured in a specific map. Another important consideration with mapping is that the specimen should be polished. An x-ray map only shows changes in x-ray intensity. That intensity change could be the result of compositional differences but it can also be caused by topographical changes in the specimen surface. Therefore, the surface texture will dominate x-ray maps when mapping unpolished surfaces, resulting in inaccurate interpretation.

Although x-ray mapping is most commonly applied to concrete analysis, x-ray microanalysis is also very useful. The process of microanalysis requires excitation of a phase within the concrete microstructure by the incident electron beam. The x-rays generated will be characteristic of the elements present in the phase and can be used to identify the elements present. This is referred to as qualitative x-ray microanalysis. The intensity of any given x-ray can be measured and the intensity will be proportional to the concentration of that element. By measuring the intensity of a given element, and comparing it to the intensity of a known standard, the concentration of any given element can be calculated. This is referred to as quantitative x-ray microanalysis. Quantitative analysis allows for the absolute identification of phases within the concrete. This is contrasted with x-ray mapping where common occurrences of elements are noted but no measure of elemental concentration is produced.

Methods for Preparing SEM/EDS Specimens

Most of the steps necessary to prepare a specimen for the SEM are identical to those presented in section B.3 of this appendix. A recent publication details the methods required for sample preparation (Stutzman and Clifton 1999). Some additional steps may have to be employed beyond the sectioning, grinding, and polishing performed to prepare an OM specimen. Most importantly, for specimens being examined in a CSEM, a conductive layer needs to be deposited on the surface of the specimen being viewed. This conductive coating is used to conduct away beam electrons after the incident electron beam strikes the surface. Without the conductive path provided by this deposited layer, a space charge develops on the specimen surface and the incident beam is displaced erratically, often scrambling the image beyond recognition. The coating process will be discussed later. If an ESEM or LVSEM is being employed, the coating process is usually unnecessary as the residual gas in the specimen chamber dispels the surface charge through ionization.

If staining is used, make sure it is done on a specimen not intended for x-ray microanalysis in the SEM as metal ions in the stain may interfere with the analysis. As an example, when using the SEM/EDS for analysis of uranyl acetate stained gels, the uranium M-*series* x-ray lines obliterate the potassium K-*series* x-ray lines, making identification of potassium in the gel impossible. Therefore, qualitative ED microanalysis of ASR gels must be performed on nonstained concrete.

Broken Pieces

Broken pieces make excellent SEM specimens and, if available, the ESEM/LVSEM is the preferred way of examining these specimens. The excellent depth of field obtained on the SEM allows for easy inspection of secondary deposits in concrete. Qualitative microanalysis on broken surfaces is also very useful and will be discussed in a later section. Before placing the broken surface in the SEM, dust off loose fragments with dry compressed air.

Polished Slabs

Polished slabs are certainly excellent SEM samples if the SEM being used allows for large specimen entry. Some SEMs are limited to specimens of only 25 to 40 mm in diameter resulting in the need for preparing small plug samples. These are sectioned and polished using methods similar to those used in preparing slabs. Polished slabs can make excellent specimens for performing microanalysis given the large, smooth surface area available for inspection.

Thin Sections

Thin sections can also be viewed in the SEM providing no cover slip has been used. Often, thin sections for SEM use are prepared thicker than normal and with a fine polish on the exposed side. These specimens are excellent for quantitative microanalysis. Epoxy impregnation will not affect the SEM image.

Specimen Coating

All specimens require a conductive coating if examined in the high vacuum environment common with the CSEM. Usually no coating is required for imaging in the low vacuum environment of the ESEM or LVSEM. Once a specimen has been coated for viewing in the SEM, it may be unusable for other analysis techniques. Therefore, SEM is usually done last if a CSEM is being used. When a conductive coating is required, there are essentially two methods and every lab should plan on obtaining both.

- Carbon/metal evaporator.
- Metal sputtering.

In carbon or metal evaporation, the specimen is placed in a vacuum chamber along with the coating material and a device capable of heating the coating material to its vaporization temperature. A high vacuum is obtained (10^{-5} torr) and the coating material is evaporated, coating the entire vacuum chamber and everything in it, including the sample. In a sputtering process, a cathode of the coating material is placed facing a steel or aluminum plate acting as an anode. The sample is placed on the anode plate and a relatively low vacuum is obtained (10^{-1} to 10^{-2} torr). Residual gas atoms in the chamber are ionized and this ionization plasma between the anode and cathode causes a transfer of cathode (coating material) atoms to the anode. This deposits a thin layer of coating atoms on the specimen.

When x-ray mapping or microanalysis is to be performed, a conductive layer of carbon is preferred because the coating layer material will not appreciably affect the results of analysis. On the other hand, metal conductive layers do affect the x-ray data. With regards to microanalysis, the metallic coatings interfere in two ways. First, x-rays generated in the conductive coating appear as peaks in the measured spectrum and may obscure other x-ray emissions from the specimen. Second, x-rays generated from phases in the specimen may be severely absorbed by the metallic coating, causing an error in calculated concentrations. However, metal coatings are good for imaging as the heavy metals used typically have a high electron yield.

Relative to a typical sputtering apparatus, evaporation systems are capable of producing a much better coat because of the high vacuum obtained. However, obtaining the high vacuum with a concrete specimen is very difficult and always results in cracking of the specimen. If a high vacuum is not used when attempting to evaporate carbon, a coarse grained or possibly soot type carbon layer is formed. These types of coating are of very low integrity and form poor or no conductive paths.

Often, it is necessary to leave larger concrete specimens in the vacuum evaporation chamber longer to reach an equilibrium vacuum suitable for good coating (10^{-4} to 10^{-5} torr). Note that any material placed in a CSEM must not out-gas in the sample chamber and degrade the CSEM vacuum. If this occurs, the CSEM will not function. Therefore, before being placed in the CSEM, every sample must first be capable of being pumped to a vacuum of 10^{-4} to 10^{-5} torr in an evaporator. This extensive pumping causes severe cracking in the specimen, making physical characterization of the concrete microstructure very difficult. Using an ESEM or LVSEM for imaging concrete eliminates the need for specimen coating and eliminates this destructive sample preparation requirement.

Methods for SEM/EDS Examination

Broken Pieces

Broken pieces are best observed using the secondary electron signal. This signal is very sensitive to topography and provides the highest image resolution possible. The backscattered electron signal can also be used but caution should be exercised in trying to interpret image contrast as compositional variance. Although backscattered electrons are not normally topography sensitive, large amounts of surface relief can cause significant variances in the backscattered electron signal. Quantitative x-ray microanalysis can usually not be performed on rough broken surfaces. The quantitative analysis programs assume a zero surface tilt with the incident beam striking the specimen normal to the surface. On irregular or rough surfaces, this is rarely possible. The local surface tilt can be entered in the quantitative analysis program but it is usually indeterminable. Measured x-ray intensities can vary significantly as a function of the local specimen tilt. Qualitative x-ray microanalysis can be conducted on broken surfaces.

However, the interpretation of the measured intensities should be done cautiously for the same reasons given above. Also, x-ray mapping of very rough surfaces may result in inaccurate data as the topographical variations will greatly affect the measured x-ray intensity.

Polished Slabs

These specimens are very good for backscattered imaging, secondary electron imaging, and both methods of microanalysis. Backscattered imaging is most useful as true compositional contrast can be obtained when imaging a polished surface. In addition, the plane geometry of the polished surface satisfies the geometric assumptions made by the quantitative analysis program, allowing for the highest possible accuracy in the analysis. In addition to quantitative and qualitative x-ray microanalysis, x-ray mapping can be readily performed on polished slabs.

Thin Sections

Thin sections are also readily examined in the SEM. Backscattered imaging is used almost exclusively as the specimen topography is limited and the topography seen is an artifact of the polishing process. However, secondary electron imaging must be used when examining microcracks because of the relatively high image resolution required. Quantitative and qualitative x-ray microanalysis and x-ray mapping are readily performed on thin sections of standard thickness. Quantitative x-ray microanalysis should not be performed on thin sections less than 10 microns in thickness. This is to satisfy another requirement of the quantitative analysis software. Namely, that the specimen is "infinitely thick" with respect to the incident electron beam. Most analysis software is unable to compensate for intensity variations resulting from the excitation of different volumes of material. The material volume reduction created by an extra thin section will cause a commensurate reduction in measured x-ray intensity.

B.5 X-RAY DIFFRACTION

Introduction

X-ray diffraction (XRD) can be used to identify the type and abundance of crystalline phases within a concrete microstructure. In general, x-ray diffraction results in identification of the inter-atomic distances between all atoms in a crystalline structure. For any crystalline material, this arrangement of atoms in an ordered unit cell is unique and characteristic of that crystalline phase. X-ray diffraction can be performed using a powder diffractometer method or Debye-Scherrer camera method. Both methods are based on the physical process of diffraction.

Equipment for X-ray Diffraction

The Powder X-ray Diffractometer

The powder x-ray diffractometer is a major piece of equipment consisting of a high voltage power supply, x-ray tube and shielding, goniometer, signal processing electronics, and computer. The high voltage power supply generates approximately 3000 watts of power at 60 kV. This is used to power the x-ray tube and generate the required x-ray flux. The goniometer is used to move the specimen through a range of orientations while being exposed to the x-ray flux. Simultaneously, the goniometer is moving an x-ray detector through a range of angles to measure the diffracted x-ray intensity from the specimen. The processing electronics converts the detected x-ray pulses to a digital signal for computer processing. Finally, the computer system is used to process and analyze the data

For a given diffraction scan over a specified angular range, a diffraction pattern is produced that is the combination of unique patterns generated from each constituent of the material. The pattern plots the diffracted intensity measured at each angle over the scan range and consists of sharp diffraction peaks, or lines, superimposed on a slowly varying background. For a given crystalline material, a family of lines occurs because diffraction occurs in multiple angular directions. The angular occurrence and relative heights of the diffraction lines forming a family is unique to that crystalline material and serves as a "fingerprint" of the material. These peaks occur at predictable angles and intensities and therefore most XRD computer systems have a set of cursors that can be used to identify phases in multiphase materials such as concrete.

In general, the more abundant phases will have peaks that are very intense and easily identifiable, as the intensity of diffraction is directly proportional to the amount of material diffracting. For the same reason, minor phases will have peaks that are very small, lost in the background, or in some cases convoluted with diffraction lines from other phases. Therefore, identifying secondary deposits by powder diffraction XRD is not always feasible. Unless the material of interest is present in quantities greater than 1 percent by volume, it will be very difficult to detect. Phases occurring in quantities of 1 to 5 percent by volume may or may not be detectable, depending upon the other phases present. Also, note that XRD is used primarily to characterize crystalline materials. Therefore, amorphous gels can not be identified by XRD but they can be confirmed as amorphous.

The Debye-Scherrer Camera

One way to improve the detection of minor phases in a material is to extract the material from the matrix and analyze it separately. For powder x-ray diffraction, this is still a problem because a significant amount (e.g., 1 to 2 grams) is required to produce a usable pattern. However, the Debye-Scherrer camera method can utilize small mounts of material and produce a diffractograph that is also characteristic of the pattern. In a Debye-Scherrer camera, or powder camera, the specimen is placed stationary on a spindle in the center of a right cylinder. A piece of film is placed around the circumference of the cylinder and the impinging x-ray flux is collimated on to the specimen. Diffraction occurs and the individual diffraction peaks appear as lines on the film. This is actually the source of the term "lines" used for referring to diffraction or spectrographic peaks. After the film is exposed, developed, and laid flat on a light table, the peak position is determined by measuring from the entrance point of the incident beam.

This technique requires a high voltage power supply, x-ray tube, shielding, and a Debye-Scherrer camera. It is easily maintained and considerably lower in cost than a powder diffractometer. However, it is best suited for analyzing single phase powders. When multiple phases are present, the overlapping lines are very difficult to interpret. Also, the exposure, developing, and interpretation can be very time consuming. This method would be most applicable if a set of standard patterns were available for concrete phases.

Methods for X-ray Diffraction

Specimens for XRD usually are in the form of a packed powder. If a phase can be physically extracted from the concrete, it can be analyzed separate from the matrix phases. Often this not possible. A sample of concrete can be ground to pass a 100-mesh sieve in any standard laboratory grinder. This is fine enough for packing into the respective specimen holders. However, peaks from minor phases will most probably not be seen when analyzed with concrete as a whole. Extracting minor phases is preferred when they are of interest.

Powder XRD

The automated powder diffractometer has the advantage of fully automatic data collection, diffraction pattern analysis software, and peak identification cursors. It has the disadvantage of not being sensitive to phases having low concentration because the minor peaks are obliterated by the peaks from major phases. To perform an analysis, it is usually necessary to specify a scan range, step size, and dwell time. The scan range controls the angular range of Bragg angles examined from x-ray diffraction. A range of 5° to 75° for the theta rotation is adequate for most needs. The step size and dwell can be any value within the ranges allowed by the manufacturer. The step size is the change in angle between two successive steps of the drive motor on the XRD goniometer. A smaller step size provides better peak resolution but longer analysis times. Dwell time is the time spent measuring the diffracted x-ray intensity at each step of the goniometer. A longer dwell time will improve the sensitivity for phases present in low concentration but will increase analysis times. Be very skeptical of any software for automatically identifying peaks in an XRD pattern. This type of software usually misidentifies most phases in a material when more than two or three phases are present. Identify all phases by manual indexing of the pattern.

Powder Camera

The powder camera is useful for analyzing small amounts of sample. However, the pattern resulting from multiple phase mixtures is often not discernible. If this method is to be used, try to establish a set of comparison patterns from materials of known composition.

APPENDIX C: LABORATORY DATA COLLECTION FORMS, FLOWCHARTS, AND DIAGNOSTIC TABLES

C.1 LABORATORY DATA COLLECTION FORMS

To provide a uniform basis for approaching concrete analysis, the following data sheets are provided. The data sheets are organized into eight major groupings with a variable number of forms provided within each group. Analysts are expected to enter data and observations on these forms. A copy of these forms accompanies each core, following it through the analysis process. Additionally, a copy of the completed forms should be provided to the engineer along with the final laboratory report. The groupings and individual forms as presented in Guideline II are as follows:

Group 1: Core Receipt and Cataloging
- Laboratory Log of PCC Pavement Cores.
- Core Sampled for Laboratory Analysis.

Group 2: Visual Inspection
- General Condition of Concrete.

Group 3: Stereo Optical Microscope Examination
- Observations of the Concrete.
- Alterations of the Aggregates.
- Results of ASTM C457.

Group 4: Staining Tests
- Summary of Staining Tests.

Group 5: Petrographic Optical Microscope Examination
- Observations of the Concrete.
- Alterations of the Aggregates.

Group 6: Scanning Electron Microscope Examination
- General Conditions.
- Summary of SEM Analysis.

Group 7: Chemical Tests
- Summary of Chemical Tests.

Group 8: X-ray Diffraction Analysis
- Analytical Conditions.

LABORATORY LOG OF PCC PAVEMENT CORES

Sheet ____ of ____

Project Designation: ______________ State: ____________________ Highway: _____________
Nearby City and Distance: ___________________________________ Direction: _____________
No. of Through Lanes (in direction sampled): ____________________ Lane Sampled: _________
Beginning Milepost/Station: ____________________ End Milepost/Station: __________________

Operator: ___________________________ Core Diameter: ______________________________
Coring Date: _________________________ Core Barrel Tip Type: _______________________
Job ID: _____________________________

Note: Each column shown below should be used to record information for all cores/pieces extracted from a single panel. "Depth" should be measured from the pavement surface to the bottom of the core/piece and recorded to the nearest 2 mm. Front direction is the direction of traffic.

CORE NUMBER	A	B	C	D	E	OTHER
CORE ID						
LOCATION/STATION						
Offset From Joint						
Offset From Shoulder						
Number of Pieces?						
Core Piece No. 1						
Position ***(Left, Right, Back, Front of core)***						
Depth (mm)						
Core Piece No. 2						
Position ***(Left, Right, Back, Front of core)***						
Depth (mm)						
Core Piece No. 3						
Position ***(Left, Right, Back, Front of core)***						
Depth (mm)						
Core Piece No. 4						
Position ***(Left, Right, Back, Front of core)***						
Depth (mm)						
REMARKS						

Analyst		Core taken by	Date Cored
Date	Core ID	Job ID	

CORE SAMPLED FOR LABORATORY ANALYSIS	
Location (circle one): A B C D E Other?:	Picture: Top View
Core Diameter: mm	
No. of Pieces:	
Ht. Piece #1: mm	
Wt. Piece #1: kg	
Ht. Piece #2: mm	
Wt. Piece #2: kg	
Ht. Piece #3: mm	
Wt. Piece #3: kg	
Ht. Piece #4: mm	
Wt. Piece #4: kg	
Total Ht.: mm	
Total Wt.: kg	
Sketch	Picture: Side View

Analyst	Job ID
Date	Core ID

Visual Inspection - General Condition of Concrete

Attach additional documentation and photographs as needed.

Diagnostic Feature	*Options*		*Comments*
Ring when struck lightly with a hammer?	Yes	No	
Does it break with your fingers?	Yes	No	
Is the concrete well consolidated?	Yes	No	
Is segregation apparent?	Yes	No	
Orientation/parallelism of aggregates?	Yes	No	
Visible surface deposits or exudate?	Yes	No	
Are cracks apparent in the paste?	Yes	No	
Are the cracks widespread?	Yes	No	
Cracks through aggregates?	Yes	No	
Are there cracks around aggregates?	Yes	No	
Deposits in cracks?	Yes	No	
Embedded items present?	Yes	No	
Adequate cover over the embedded items?	Yes	No	
Are embedded items corroded?	Yes	No	
Underside voids on elongated aggregates?	Yes	No	
Are air voids filled?	Yes	No	
Does paste hardness seem normal?	Yes	No	
Is paste hardness uniform throughout?	Yes	No	
High paste content?	Yes	No	
Gradation of aggregates?	gap	uniform	
Coarse aggregate top size?			
Coarse aggregate type?	crushed	natural	
Coarse aggregate rock type?			
Fine aggregate type?	crushed	natural	
Fine aggregate rock type?			
Alteration/reaction with aggregates?	Yes	No	

Comments____

Analyst	Job ID
Date	Sample ID

Stereo OM - Observations of the Concrete

Attach additional documentation and micrographs as needed.

Diagnostic Feature	**Options**	*Comments*
Bleeding?	bleed channels pockets under aggregates	
Fractures?	none through aggregates in paste around aggregates	
Air void infilling?	yes partial no	
Air void shape	spherical irregular ellipsoidal	
Embedded item condition?	good corroded	
Surface condition?	cracked carbonated weathered leached normal	
Aggregate reaction products?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste	
Gaps around coarse or fine aggregates?	description estimate occurrence	
Gap widths larger for larger particles?	yes no	
Are the gaps filled?	yes no	
Coarse aggregate type?	gravel quarried crushed gravel slag other	
Lithological types of coarse aggregate?	specify types identified	
Orientation/parallelism of coarse aggregate?	specify which and specify direction of orientation	
Fine aggregate type?	natural other *(specify)* crushed	
Lithological types of fine aggregate?	specify types identified	
Paste color – *note uniformity of color.*	white light gray dark gray yellow green other (*specify)*	
Paste hardness – *note uniformity of hardness.*	soft hard medium	

Comments ____

____

Analyst	Job ID
Date	Sample ID

Stereo OM Observations - Alterations of the Aggregates

Attach additional documentation and micrographs as needed.

Coarse Aggregates

Diagnostic Feature	*Options*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution or softening of aggregates?	yes no	
Reaction rims?	yes no	
Reaction products from alteration?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	

Fine Aggregates

Diagnostic Feature	*Options*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution or softening of aggregates?	yes no	
Reaction rims?	yes no	
Reaction products from alteration?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	

Narrative Description of Alteration ____

____

Analyst	Job ID
Date	Sample ID

Results of ASTM C 457

Method Used:	Length Traversed
	Area Traversed
Magnification	No. of Stops

Air-Void System Parameters

Specification	Typical Range for Acceptable Air Entrained Concrete	Calculated Value for Original Air-Void System *(infilling counted as air)*	Calculated Value for Filled Air-Void System *(infilling counted as paste)*
Spacing Factor ($\bar{L}$)	0.01 - 0.02 mm		
Specific Surface (a)	23.6 - 43.3 mm^{-1}		
Paste/Air Ratio	4 - 10		
Void Frequency (n)			

Phase Abundance Analysis

Phase	**Volume Percent**
Coarse Aggregate	
Fine Aggregate	
Paste	
Original Air Content	
Filled Voids	

Comments____

Analyst	Job ID
Date	

Sulfate/ASR Reaction Product Staining

Attach additional documentation and micrographs as needed.

Core ID	*Method Used*	*Positive Staining*	*Comments*
		Yes No	
		Yes No	
		Yes No	
		Yes No	

Depth of Carbonation

Attach additional documentation and micrographs as needed.

Core ID	*Depth of Carbonation*	*Comments*

Comments ____

Analyst	Job ID
Date	Sample ID

Petrographic OM - Observations of the Concrete

Attach additional documentation and micrographs as needed.

Diagnostic Feature	*Selected Descriptors*	*Comments*
w/c in bulk?	specify method and value	
w/c at surface if different than bulk?	specify method and value	
Evidence of trapped bleed water?	water voids below horizontal aggregate faces	
Air-void structure at surface intact?	specify	
Calcium hydroxide depletion?	specify	
Sub parallel cracking or delamination at surface?	specify	
Paste density variations around the aggregates?	specify	
Cracking?	cracks through aggregates cracks around aggregates cracks in paste	
Secondary deposits?	yes no	
Location of secondary deposits?	air voids cracks through aggregates cracks around aggregates cracks in paste	
Identify deposits.	specify	
Lithological details of coarse aggregate?	specify	
Lithological details of fine aggregate?	specify	
Mineral admixtures?	present not present	
Identification	fly ash silica fume GGBF slag other	

Narrative Description of Petrography Results: ______________________________

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Analyst	Job ID
Date	Sample ID

Petrographic OM Observations - Alterations of the Aggregates

Attach additional documentation and micrographs as needed.

Coarse Aggregates		
Diagnostic Feature	*Selected Descriptors*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution of aggregate?	yes no	
Reaction rims?	yes no	
Reaction products?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	
Identify reaction products	specify	

Fine Aggregates		
Diagnostic Feature	*Selected Descriptors*	*Comments*
Degree of alteration?	isolated extensive moderate	
Cracking in aggregates?	yes no	
Internal cracks narrow from center of aggregate out?	yes no	
Cracks through aggregates extend into the paste?	yes no	
Dissolution of aggregate?	yes no	
Reaction rims?	yes no	
Reaction products?	yes no	
Location of reaction products?	air voids cracks through aggregates cracks around aggregates cracks in paste pockets adjacent to aggregates	
Identify reaction products	specify	

Narrative Description of Alteration ______________________________

____

____

____

Analyst	Job ID
Date	

SEM - General Conditions	
SEM or CSEM	Operating Pressure
Samples Conductive Coated (Y/N)	Coating Method/Thickness
Samples Dehydrated (Y/N)	Dehydration Method

SEM - Conditions for Quantitative Microanalysis	
SEM or CSEM	Operating Pressure
Accelerating Voltage	Beam Current
Working Distance	Standardless or full-quantitative (with standards) analysis?
Oxygen measured or determined by stoichiometry?	Analysis done by EDS or WDS?

SEM - Conditions for X-ray Mapping	
SEM or CSEM	Operating Pressure
Accelerating Voltage	Beam Current
Working Distance	Map Resolution
Map Dwell Time	Elements Mapped

Comments ____

Analyst	Job ID
Date	Sample ID

Summary of Scanning Electron Microscope Analysis

- Summarize observations and hypotheses. Note micrograph numbers and spectrum identifications when referenced. Attach all data in a folder labeled with the core number and the title SEM Data.

Analyst	Job ID
Date	

Summary of Chemical Tests

Determination of w/c

Specimen ID	Method Used	Measured w/c	Comments

Determination of Sulfate Concentration

Specimen ID	Method Used	Sulfate Concentration	Comments

Determination of Chloride Concentration

Specimen ID	Method Used	Chloride Concentration	Comments

Analyst	Job ID
Date	

XRD - Analytical Conditions	
Type of specimen (e.g., powder, slab)	Specimen mounting method? (Side drifted, pressed pellet)
X-ray tube kV and mA	X-ray tube target material
Was a primary beam filter used and what type?	Monochromater used? (Y/N)
Divergence slit (specify mm or degrees)	Receiving slit (specify mm or degrees)
Scan range (degrees 2θ)	Scan rate (degrees/min)
Dwell time (seconds/step)	Step size (degrees/step)
Peaks identified by automatic or manual search (auto or manual)?	Background subtracted before analysis?

XRD - Results of Qualitative Analysis

- Summarize observations and hypotheses. List phases identified. Note diffraction pattern identifications when referenced. Attach all data in a folder labeled with the core number and the title Qualitative XRD Data.

C.2 FLOWCHARTS AND DIAGNOSTIC TABLES

This section contains copies of the flowcharts and diagnostic tables for reproduction. The flowcharts present a systematic method for diagnosing MRD in concrete pavements. The analyst inspects the concrete using the methods described in Guideline II, being guided by the hierarchy of questions presented in the flowcharts presented in figures C-1 through C-4 in this appendix (figures II-15 through II-17 in guideline). The responses to the questions presented in the flowcharts determine what analytical procedures will be performed. As the analyst moves through the flowcharts, there is the potential for more than one MRD being identified. The analyst needs to keep track of all possibilities identified and then use the diagnostic tables presented in tables C-1 through C-7 in this appendix (tables II-1 through II-7 in guideline) to help isolate the most likely MRD(s). The tables summarize the common diagnostic features associated with each MRD.

Possible Distress	Present		Additional Information
Error in mix proportioning	Yes	No	See recommended literature
Poor placement	Yes	No	See recommended literature
Poor finishing/curing	Yes	No	See recommended literature
Poor steel placement	Yes	No	See recommended literature
Carbonation at depths > 5-10 mm	Yes	No	See recommended literature

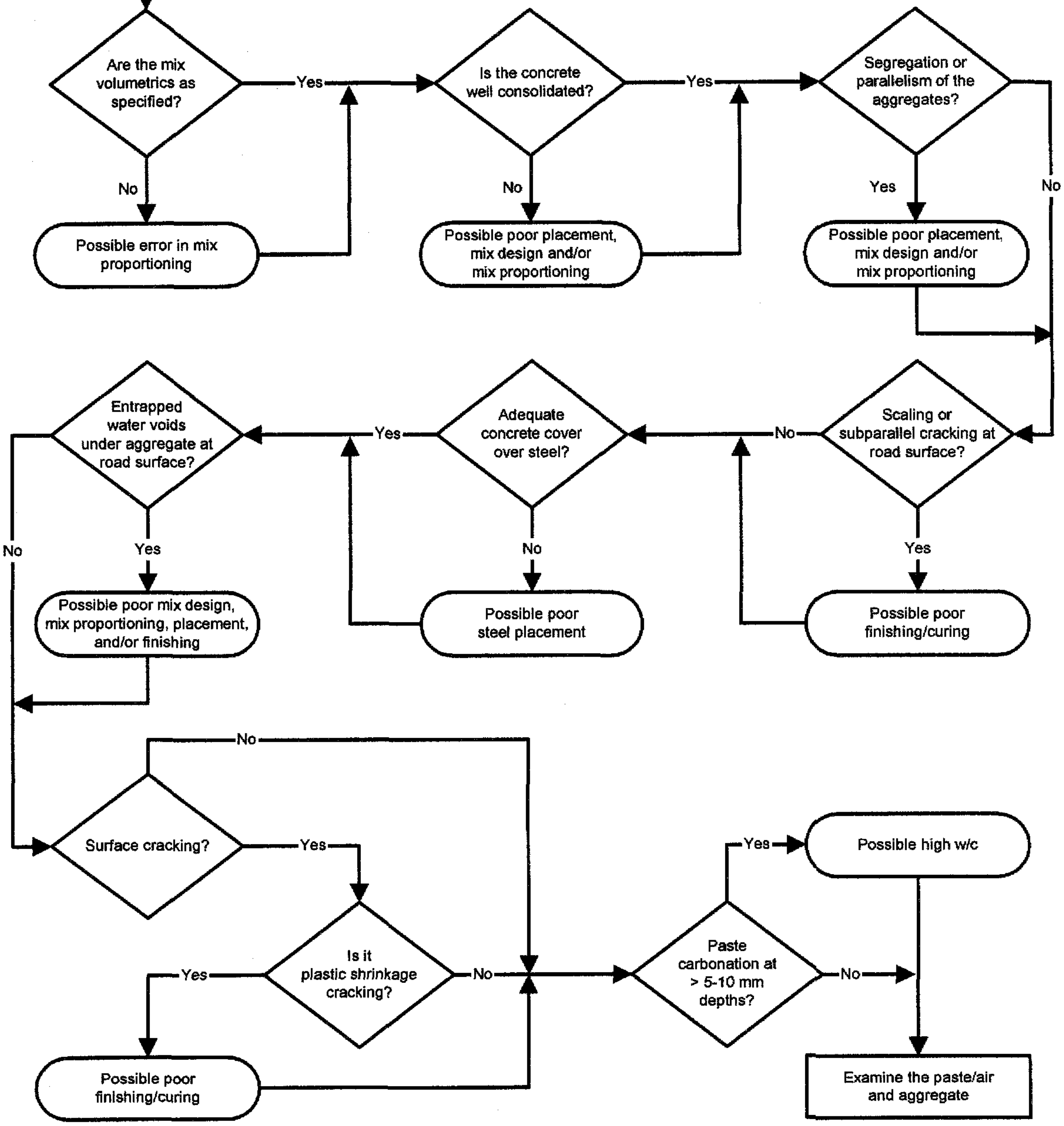

Figure C-1. Flowchart for assessing general concrete properties based on visual examination.

Possible Distress	Present		Additional Information
Shrinkage cracks or sample preparation cracks	Yes	No	See recommended literature
Corrosion of embedded steel	Yes	No	Table C-1
Paste freeze-thaw	Yes	No	Table C-2
Aggregate freeze-thaw	Yes	No	Table C-3
Sulfate attack	Yes	No	Table C-4
Deicer attack	Yes	No	Table C-5
Infilling material	Yes	No	Figure C-3

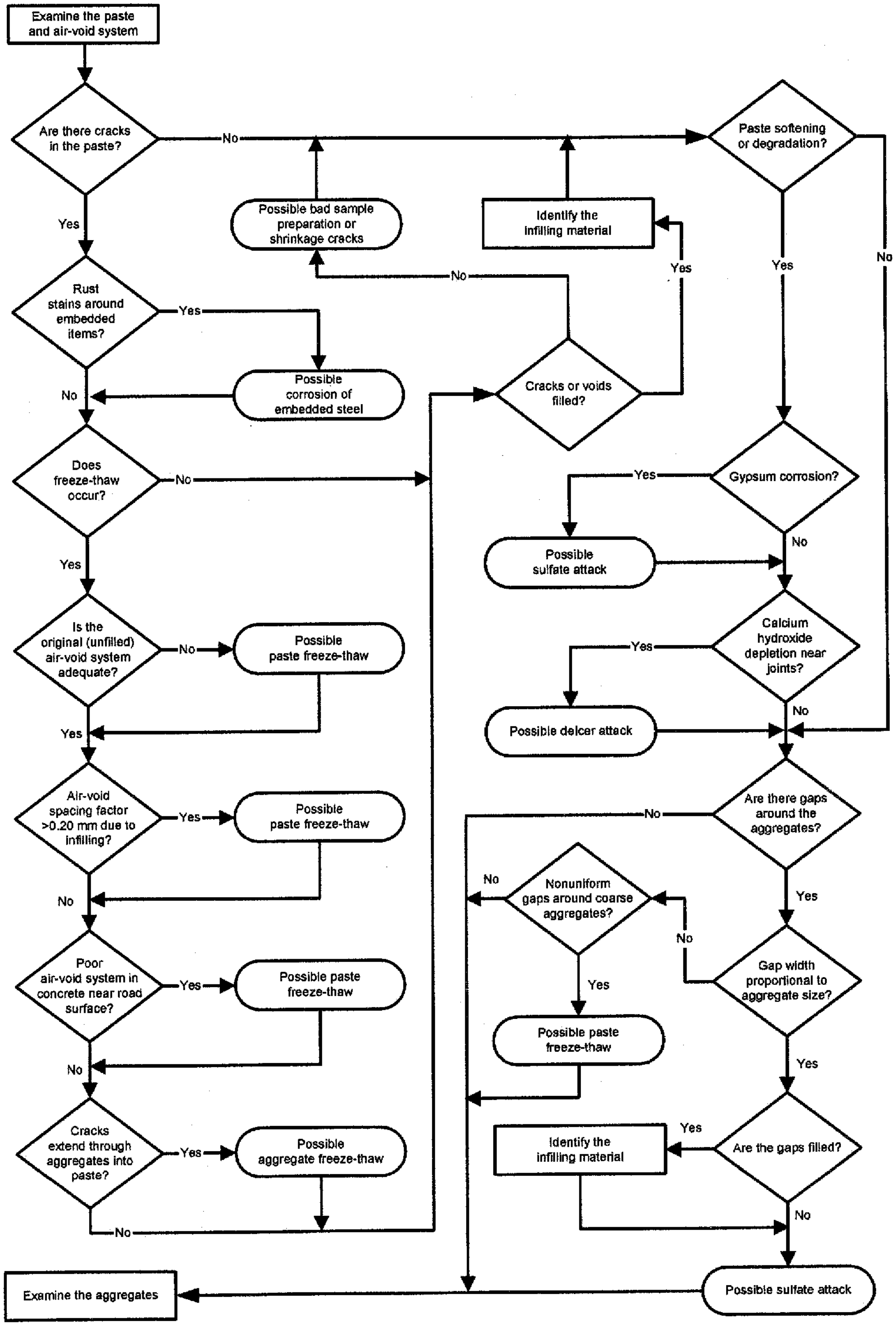

Figure C-2. Flowchart for assessing the condition of the concrete paste and air.

Possible Distress	Present		Additional Information
Corrosion of embedded steel	Yes	No	Table C-1
Sulfate attack	Yes	No	Table C-4
Deicer attack	Yes	No	Table C-5
Alkali–silica reaction	Yes	No	Table C-6
Alkali–carbonate reaction	Yes	No	Table C-7

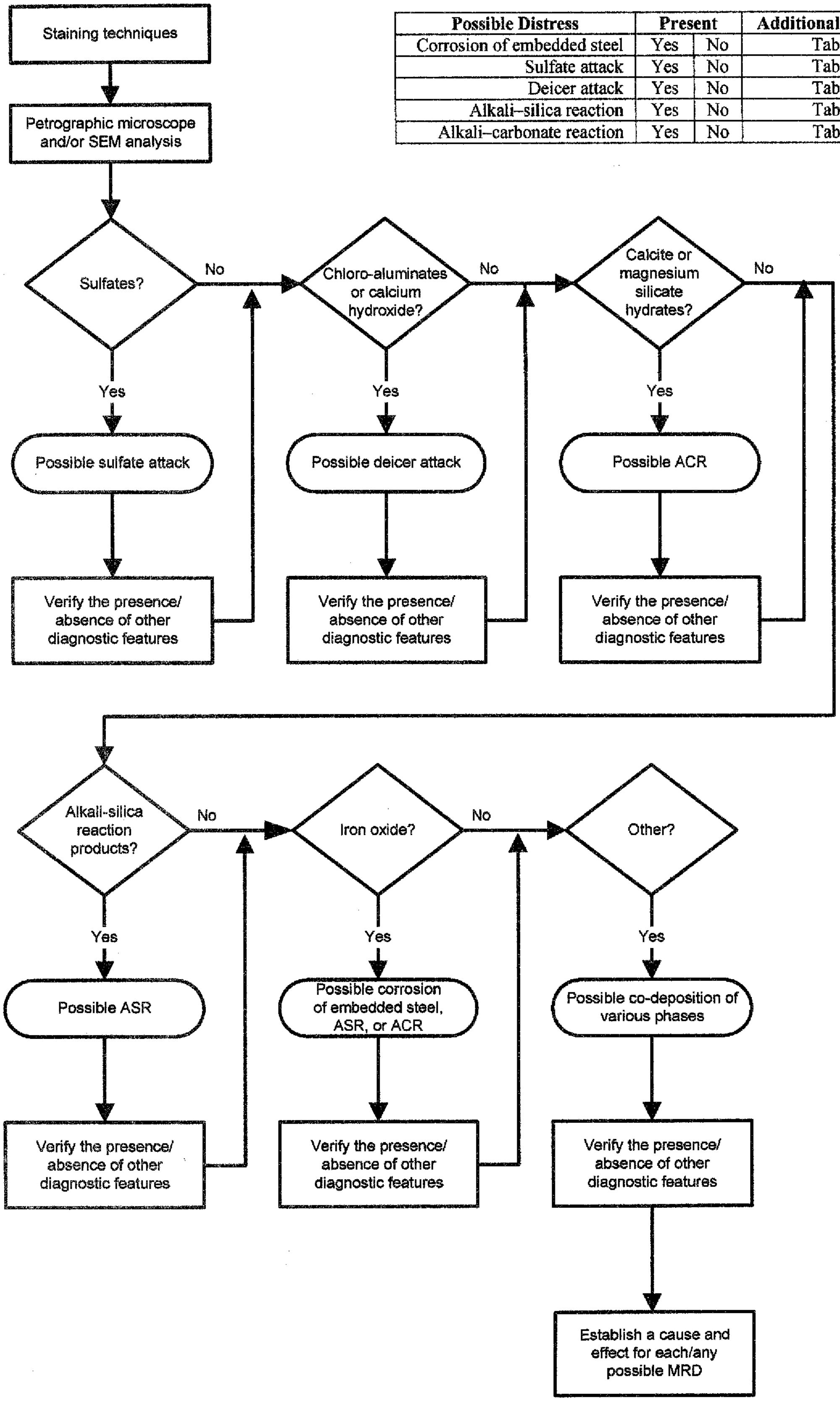

Figure C-3. Flowchart for identifying infilling materials in cracks and voids.

Possible Distress	Present		Additional Information
Natural cracking of aggregate	Yes	No	See recommended literature
Sample preparation cracks	Yes	No	See recommendedlLiterature
Aggregate freeze-thaw	Yes	No	Table C-3
Natural weathering of aggregates	Yes	No	See recommended literature
Alkali–silica reaction	Yes	No	Table C-6
Alkali–carbonate reaction	Yes	No	Table C-7
Infilling material	Yes	No	Figure C-3

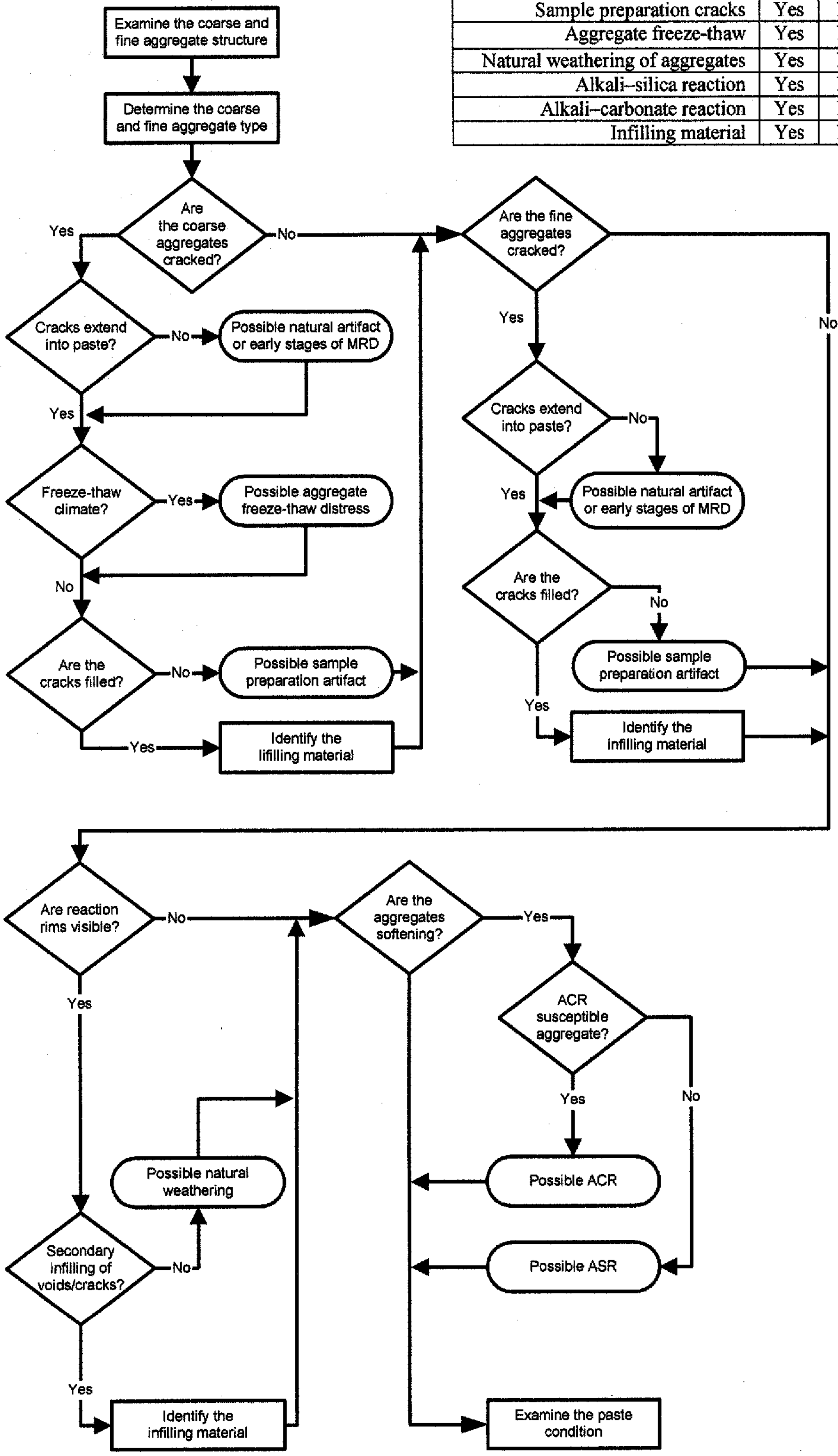

Figure C-4. Flowchart for assessing the condition of the concrete aggregates.

Table C-1. Diagnostic features of corrosion of embedded steel.

Diagnostic Feature	Method of Characterization	Comments
Spalling and delamination of concrete over reinforcing steel	Field Evaluation Visual Inspection	Visual inspections can be used to readily identify areas affected by corrosion of embedded steel. It is characterized by rusting steel at the bottom of the spalled out area and rust stains on the loose pieces.
Visible corrosion products	Field Evaluation Visual Inspection Stereo OM Petrographic OM	The "rust" seen may contain crystalline magnetite but is primarily amorphous.

Table C-2. Diagnostic features of paste freeze-thaw damage.

Diagnostic Feature	Method of Characterization	Comments
Surface scaling or subparallel cracking	Field evaluation Visual	Look for loss of paste at road surface, exposed coarse aggregate, and/or scaling. Can be isolated to surface or through slab depth.
	Stereo OM Petrographic OM	Look for delamination/ subparallel cracking at surface or evidence of an overworked surface such as decreased air content at the surface.
Inadequate air-void system	Stereo OM	Measure air-void parameters consistent with ASTM Method C 457. Typical parameters for good concrete are as follows: Spacing Factor ($\overline{L}$) = 0.1 - 0.2 mm Maximum Spacing ($\overline{L}$) = 0.2 mm Typical Specific Surface (α) = 23.6 - 43.3 mm^{-1} Paste/Air Ratio - 4 - 10 The air-void size distribution should also be noted as different size distributions can yield similar values of $\overline{L}$ and α.
Secondary deposits filling air voids	Staining	Deposits stained cannot be analyzed by any other method to determine their composition.
	Stereo OM	Deposits result as water freely moves through the distressed paste. In extreme cases, the air-void system may be further compromised when significant numbers of voids are filled with secondary deposits. Common deposits include calcium hydroxide, calcium carbonate, ASR reaction products, and various sulfates including ettringite.
	Petrographic OM	Secondary deposits are commonly identified using the petrographic OM.
	SEM	The SEM operated at high vacuum is very useful for determining the composition of secondary deposits. Direct output of phase composition allows for absolute identification.
Microcracking around aggregates	Stereo OM Petrographic OM	Cracking will be in the paste. If cracks pass through aggregates, check table C-3 for coarse aggregate freeze-thaw, table C-6 for ASR, and table C-7 for ACR. Cracks will fill with secondary deposits.
	LVSEM	Severe cracking in the paste occurs if PCC is observed in a CSEM. Hydration products and ASR reaction products dehydrate in the high vacuum in the CSEM.

Table C-3. Diagnostic features of aggregate freeze-thaw deterioration.

Diagnostic Feature	Method of Characterization	Comments
Cracking near joints/cracks Staining/Spalling	Field evaluation	Has a very characteristic cracking pattern concentrated at corners, joints, and cracks (SHRP 1993). Increased permeability results from the cracking. Calcium hydroxide is leached and re-deposits on surface where it carbonates.
Cracks through nonreactive coarse aggregates	Visual inspection Stereo OM Petrographic OM SEM	Cracks through nonreactive coarse aggregates are very typical of D-cracking. Be very careful to completely rule out alkali–aggregate reaction. See tables C-6 and C-7 for diagnostic features of ASR or ACR.
Nonuniform gaps around coarse aggregates	Visual inspection Stereo OM	Gaps between the aggregate and paste form. These gaps may result from the dissolution of calcium hydroxide at the aggregate/paste interface or coarse aggregate dilation due to freezing. Subsequent redeposition of calcium hydroxide or calcite may occur in the cracks. Check for sulfate or ASR reaction products in cracks surrounding aggregates. A negative result helps to confirm aggregate freeze-thaw.
Known freeze-thaw susceptible aggregate Large top size aggregate	Records review Visual inspection Stereo OM SEM	Check aggregate sources for known freeze-thaw performance Aggregate freeze-thaw is more common in large aggregates (> 38 mm) and rare in aggregates smaller than 12.5 mm.
Poor void structure in the aggregate	Petrographic OM CSEM LVSEM	As a percentage of the total aggregate void space, excessive amounts of voids in the aggregate with diameters less than 5 microns is thought to be detrimental to aggregate freeze-thaw resistance.

Table C-4. Diagnostic features of sulfate attack.

Diagnostic Feature	Method of Characterization	Comments
Map cracking	Field evaluation	Paste expansion commonly results in map cracking over entire surface. In some cases, it is isolated to joints/cracks.
Deteriorated paste		Sulfate attack may result in paste "crumbling," commonly at joints. Loose aggregate observed in resulting void.
External source of sulfur only	Soil analysis Deicer analysis	Identify a sole source of sulfate that is external to the concrete to confirm external sulfate attack. Having both external and internal sources confounds the diagnosis.
Internal source of sulfur only	Records review SEM XRD	Identify a sole source of sulfate that is internal to the concrete to confirm internal sulfate attack. Having both external and internal sources confounds the diagnosis.
Paste expansion	Stereo OM Petrographic OM SEM	Expansion occurs, usually over a large area. Gaps form around aggregates with the gap width proportional to the aggregate diameter.
Significant sulfate deposits in cracks and voids	Staining Stereo OM	Stained deposits or stained paste cannot be accurately analyzed by SEM to determine their composition. A common sulfate deposit is ettringite. This is commonly recognized by acicular needle-like crystals infilling voids and cracks.
	Petrographic OM	Common sulfate deposits can be identified. Mixtures with other phases may be more difficult to identify.
	SEM	All deposits are readily identified using the SEM in high vacuum mode. Co-deposition with other phases may be more closely studied.
Significant sulfate deposits in the cement paste	Petrographic OM	Fluorescent dye epoxy impregnation assists in identifying microcracks in the paste. Cracks unfilled with epoxy should be assumed were created during sample polishing.
	LVSEM/ESEM	Cracks resulting from sulfate expansion can be viewed in an LVSEM/ESEM but caution should be exercised in identification of micron scale microcracks. Even in an ESEM, some dehydration does occur, leading to possible cracking. Cracks unfilled with epoxy should be assumed were created in the SEM or during sample polishing. If viewing an unimpregnated specimen, cracks unfilled with secondary deposits should be assumed were created in the SEM or during sample preparation.
Microcracking	Stereo OM Petrographic OM	For filled cracks, the cracks may have been present from other distress and secondary deposits formed in the cracks. Fluorescent dye epoxy impregnation greatly improves the identification of microcracks in the paste. Cracks not filled with epoxy are probably artifacts of sample preparation.
	SEM	A characteristic spectrum for dehydrated ettringite has approximate element ratios of 1:2:4 (Al:S:Ca) by weight.

Table C-5. Diagnositc features of deicer scaling/deterioration.

Diagnostic Feature	Method of Characterization	Comments
Staining at joints or cracks	Field evaluation	Staining results from calcium hydroxide depletion and subsequent carbonation at surface.
Scaling or crazing of slab surface	Field evaluation Visual inspection Stereo OM	Common visual diagnostic feature. Similar and possibly related to paste freeze-thaw damage. See table C-2 for more on paste freeze-thaw damage.
Calcium hydroxide depletion near joints	Stereo OM Petrographic OM SEM	Calcium hydroxide (CH) is most soluble near the freezing point of water. Cyclic freezing and thawing from repeated deicer applications can accelerate the dissolution of CH near joints/cracks.
Secondary deposits of chloroaluminates	Petrographic OM SEM	Chloride ions released from dissolved salts can form these phases with aluminate phases in the paste.

Table C-6. Diagnostic features of ASR.

Diagnostic Feature	Method of Characterization	Comments
Map cracking with or without exudate Evidence of pavement expansion	Visual inspection	ASR is characterized by widespread map cracking. Can be more severe at joints and may be preferentially oriented perpendicular to the direction of least restraint (e.g., in pavement slabs, longitudinal cracks often predominate). Exudate common but not always observed. Evidence of joint closing or shoving of shoulder or fixed structures are possible indicators of expansion.
ASR reaction product in cracks and voids	Stereo OM Petrographic OM	A glassy clear to white amorphous reaction product resulting from an alkali–silica reaction. ASR reaction product is found within reacted particles, cracks, and air voids. The presence of ASR reaction product alone does not indicate ASR distress, as it must be of sufficient volume and composition to cause deleterious expansion.
	SEM	ASR reaction product can be chemically characterized with the SEM operating at a high vacuum. Primarily high alkali (low calcium) ASR reaction products are expansive.
Reaction rims on aggregates	Visual inspection Stereo OM Petrographic OM SEM	Reaction rims are often seen on most reactive aggregate. Reaction rims are common on aggregates that are undergoing ASR. Good gravel aggregates can exhibit rims that appear similar to ASR reaction rims. These are typically the result of weathering. Reaction products present help confirm ASR.
Aggregate known to be reactive	Records review	Check to see if the aggregates used were from a source that is known to be reactive.
Microcracking radiating from reacted cracked aggregates Softening of the aggregate	Visual inspection Stereo OM Petrographic OM LVSEM/ESEM	Reacted aggregates may break down internally and often partially dissolve. As the aggregate degrades, the ASR reaction product produced may be expansive and cause cracking to occur. The cracks are within the periphery of the aggregate but around the center. Often the cracks will narrow from the center of the aggregate out. Coarse and fine aggregates can both cause ASR distress. Common reactive aggregates are composed of or include chert, flint, siliceous shale, strained quartz, and porous volcanic glasses.

Table C-7. Diagnostic features of ACR.

Diagnostic Feature	Method of Characterization	Comments
Map Cracking with or without exudate Evidence of expansion	Visual inspection	ACR is characterized by widespread map cracking. Can be more severe at joints. Exudate common but not always observed. Evidence of joint closing or shoving are possible indicators of expansion.
Cracks radiating from the coarse carbonate aggregate into the paste	Visual inspection Stereo OM Petrographic OM LVSEM	Deterioration from ACR results from the expansion of the aggregate that causes cracks in the aggregate, which propagate into the paste. The expansion is the result of reaction products produced in the dedolomization reaction.
Aggregate known to be reactive	Records review	Check to see if the aggregates used were from a source that is known to be reactive.
Characteristic texture of ACR reactive aggregates	Petrographic OM	Most ACR reactive aggregates have a characteristic texture. ASTM C 856 states the basic texture as being relatively larger rhombic dolomite crystals in a fine-grained calcite matrix with clay and silt-sized quartz. Substantial amounts of both dolomite and calcite are present. Other textures have been reported as reacting, with a common thread being soluble magnesium phases that react to form expansive products.
	CSEM LVSEM	Calcium and magnesium silicate hydrates are common reaction products.

APPENDIX D: REFERENCES

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