ENSURING DURABILITY OF CONCRETE PAVING MIXTURES
PART I: MECHANISMS AND MITIGATION

INTRODUCTION AND BACKGROUND

Hydraulic cement concrete (HCC, but hereafter referred to simply as concrete) is composed of aggregates bound together by a hydrated cement paste (HCP). Concrete is an attractive material for the construction of pavements because it is readily available, affordable, and is known for longevity. The structural performance of concrete pavement is addressed through consideration of the system's response to repeated loading, factoring in material properties, support conditions, slab geometry, load transfer, and climatic impacts. The assumption inherent in pavement structural design is that if the concrete possesses the required mechanical properties, the pavement will achieve design expectations as long as the concrete is durable. But durability is not an intrinsic, measurable property of concrete. Instead it is a set of material properties that are required for the concrete to resist the particular environment in which it serves (TRB 2013). The same concrete placed in a mild, dry environment may remain wholly intact for decades yet rapidly disintegrate if exposed to chemical deicers in a wet, freeze-thaw environment. Both the environment and materials must be considered together to specify and construct durable concrete pavements.

This Tech Brief presents various distresses that can develop because of issues related to durability (commonly referred to as materials-related distress [MRD]) and describes strategies to prevent or mitigate each type of MRD. MRD can compromise the integrity of concrete and result in costly repairs and ultimately failure of a concrete pavement, even if the structural design is sound. A second Tech Brief (titled *Ensuring Durability of Concrete Paving Mixtures-Part II: Testing and Construction*) focuses on test methods and construction techniques that can be utilized to reduce the risk of MRD.

MRD MECHANISMS

MRD mechanisms are commonly broken into physical and chemical mechanisms, as shown in tables 1 and 2, respectively. A number of excellent resources are available that discuss these mechanisms in detail, including Van Dam et al. (2002a; 2002b); Sutter et al. (2006); Taylor et al. (2006); ACI (2008); Jones et al. (2013); Thomas, Fournier, and Folliard (2013); Sutter (2015); and Taylor and Wang (2015).

Regardless of the mechanism, the impact of climate is clear, with moisture and freeze/thaw actions implicit in a number of MRDs (particularly in the presence of deicing chemicals). Common strategies to improve the durability of paving concrete focus on properly selecting cementitious components and aggregate, reducing the permeability of the concrete, and reducing the susceptibility of the HCP to chemical attack. The following sections of this Tech Brief discuss contributing factors and mitigation strategies to improve the durability of paving concrete.

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Table 1. Summary of common concrete pavement MRD types due to physical mechanisms (based on Van Dam et al. 2002a).

Type of MRD	Observed Distress	Cause	Time of Appearance
Freeze-Thaw Deterioration of Hardened Cement Paste	Crazing or surface scaling, or joint spalling or deterioration. Generally initiates near joints or cracks; possible internal disruption of concrete matrix.	Deterioration of HCP due to repeated freeze-thaw cycles in a saturated state. Entrained air-void system insufficient to protect HCP from damage.	1-10 years
Deicer Scaling/ Deterioration	Crazing or surface scaling with possible alteration of the concrete pore system or the HCP, leading to staining at joints and cracks, followed by joint deterioration.	Deicing chemicals amplify freeze-thaw deterioration by increasing the level of saturation and pressures generated; may interact chemically with HCP (Sutter et al. 2006; Jones et al. 2013).	1-5 years
Freeze-Thaw Deterioration of Aggregate	Cracking parallel to joints and cracks, followed by spalling; may be accompanied by surface staining.	Freezing and thawing of susceptible coarse aggregates results in fracturing or excessive dilation of aggregate.	10-25 years

Table 2. Summary of common concrete pavement MRD types due to chemical mechanisms (based on Van Dam et al. 2002a).

Type of MRD	Observed Distress	Cause	Time of Appearance
Alkali-Silica Reactivity (ASR)	Pattern cracking at joints and often over entire slab surface. Exudate often accompanies cracking. May have expansion-related distresses (joint closure, spalling, blowups).	Reaction between alkalis in the pore solution and reactive silica in aggregate results in formation of an expansive gel and degradation of the aggregate particle.	5-25 years
Alkali-Carbonate Reactivity (ACR)	Map cracking over entire slab area and accompanying expansion-related distresses (joint closure, spalling, blowups).	Aggressive expansive reaction between alkalis in pore solution and certain dolomitic aggregates which commonly involves dedolomitization and brucite formation.	5-15 years
External Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area, ultimately resulting in joint or surface deterioration.	Formation of ettringite, gypsum, or thaumasite that occurs when external sources of sulfate (e.g., groundwater, deicing chemicals) react with aluminate phases in HCP.	1-10 years
Internal Sulfate Attack	Fine cracking near joints and slab edges or map cracking over entire slab area. Evidence of expansion-related distress (joint closure, spalling, blowups).	Delayed ettringite formation (DEF) from high early-age curing temperatures that results in either expansive disruption in the paste phase	1-5 years
Corrosion of Embedded Steel	Spalling, cracking, and deterioration at areas above or surrounding embedded steel.	Chloride ions penetrate concrete, facilitating corrosion of embedded steel. Increased volume of corrosion products causes distress.	3-20 years

FACTORS AFFECTING DURABILITY OF PAVING CONCRETE

Durability is not an intrinsic property of concrete. The environment in which the concrete serves largely dictate the properties it must possess to resist MRDs. The primary external factors affecting the durability of paving concrete are climatic conditions and external sources of sulfate.

Climatic Conditions

Major climatic factors that affect the durability of paving concrete are moisture and freeze-thaw cycles, particularly in the presence of chemical deicers. Moisture is a necessary component in the initiation and progression of all of the MRDs cited. For physical mechanisms, the phase transition of liquid water to ice is the primary cause of aggregate and paste deterioration. For chemical mechanisms, the chemical bonding of water in the formation of new minerals (e.g., ASR gel, ettringite, and iron oxide) leads to the damaging expansion. For concrete pavements on ground, sufficient moisture is almost always available in the environment (from above or below the slab) to facilitate the initiation and progression of an MRD.

In many climates, concrete pavements are subjected to multiple annual cycles of freezing and thawing. Partially dry concrete will not be damaged from these freeze-thaw cycles because the larger pores in the HCP are empty and provide adequate space to accommodate hydraulic, osmotic, and crystallization pressures that develop due to ice formation. But if this same concrete undergoes freezing and thawing in a critically saturated state (above approximately 85 percent saturation), damage will occur within a few cycles, irrespective of air void volume (Jones et al. 2013).

Adding to the harshness of the freeze-thaw environment is the application of chemical deicers. Applied to improve the safety of the roadway, many of these deicers can negatively impact concrete durability. Chemical deicers can physically affect concrete by increasing the degree of concrete saturation to levels that approach or exceed damaging critical saturation, by amplifying osmotic pressure through changes in the HCP pore solution chemistry, and by generating thermal shock due to the thermodynamics of melting ice. They also

contribute to the development of salt crystallization pressures, among other factors (Mindess, Young, and Darwin 2003; Kosmatka and Wilson 2011; Villani et al. 2015). All of these factors can contribute to the development of deicer-related scaling of the pavement surface.

More recently, research has focused on the potential for chemical damage from brine solutions of calcium chloride and magnesium chloride. It has been demonstrated in the laboratory that the formation of calcium oxychloride can result in significant damage to the concrete even if the concrete does not freeze (Sutter et al. 2006; Jones et al. 2013; Villani et al. 2015). This mechanism is believed to be at least partially responsible for joint deterioration observed in some midwestern states.

The American Concrete Institute (ACI) uses the freeze-thaw exposure classes presented in table 3 to describe different environmental conditions to which concrete may be exposed. The exposure becomes more severe as the concrete becomes more saturated, with the most severe conditions (Exposure Class F3) occurring when the concrete is exposed to freezing and thawing and deicing chemicals.

External Sources of Sulfate

The second exposure condition to consider is whether the concrete will be exposed to an external source of water-soluble sulfates. Although sulfates can be very damaging to concrete (ACI 2008), pavements are not commonly exposed to sulfates and, as a result, sulfate attack is somewhat rare in pavements (Sutter 2015). The external sulfate ions penetrating the concrete react negatively with HCP phases, resulting in the formation of expansive or water-soluble minerals. The external source of water-soluble sulfates is most often soils and the degree to which concrete is affected is largely dependent on the source of the sulfate ions, their concentration, the permeability of the concrete, and the chemical composition of the HCP phases.

ACI identifies sulfate exposure classes based on the concentration of water-soluble sulfate found in the soil or in water. These exposure classes are presented in table 4.

Table 3. ACI freeze-thaw exposure classes (ACI 2014).

Exposure Class	Severity	Condition
F0	Not Applicable	Concrete not exposed to freezing and thawing cycles
F1	Moderate	Concrete exposed to freezing and thawing cycles, and occasional exposure to moisture, but no deicing chemical exposure[1]
F2	Severe	Concrete exposed to freezing and thawing cycles, and in continuous contact with moisture, but no deicing chemical exposure[2]
F3	Very Severe	Concrete exposed to freezing and thawing cycles and in continuous contact with moisture and exposed to deicing chemicals[3]

1 Examples are vertical surfaces above the level of snow accumulation and horizontal, elevated floors in areas protected from direct exposure to moisture.
2 Examples are vertical surfaces below the level of snow accumulation, vertical surfaces with sufficient moisture exposure to allow the concrete to be near saturation prior to freezing, retaining walls or other vertical elements with one side exposed to moisture, and slabs-on-grade that are not protected from freezing
3 Examples are vertical surfaces that may have deicing chemical-contaminated snow piled against them, sidewalks or pavements that receive deicing chemicals, and concrete that receives frequent exposure to seawater and freezing and thawing conditions.

Table 4. Severity of exposure conditions determined from sulfates in soil or water (ACI 2014).

Exposure Class	Water-soluble sulfate (SO_4)[1] in soil, %	Sulfate (SO_4)[1] in water, ppm
S0 (Not applicable)	SO4 <0.10	SO4 <150
S1 (Moderate)	0.10 ≤ SO4 <0.20	150 ≤ SO4 <1500 Or Seawater
S2 (Severe)	0.20 ≤ SO4 ≤ 2.00	1500 ≤ SO4 ≤ 10,000
S3 (Very severe)	SO4 >2.00	SO4 >10,000

1Sulfate expressed as SO4 is related to sulfate expressed as SO3, as given in reports of chemical analysis of portland cements as follows: SO3 x 1.2 = SO4

MITIGATING SPECIFIC MRDS AFFECTING THE DURABILITY OF PAVING CONCRETE

This section presents mitigation strategies to address the specific MRDs listed in tables 1 and 2, focusing on those most commonly affecting pavements, including freeze-thaw deterioration of HCP, deicer scaling, chemical deicer attack, aggregate freeze-thaw deterioration, alkali-silica reactivity, and external sulfate attack.

Freeze-Thaw Deterioration of Hydrated Cement Paste (HCP)

The freeze-thaw durability of concrete is closely linked to the size and volume of air bubbles entrained in the concrete (ACI 2008; Kosmatka and Wilson 2011). Air-entraining admixtures (AEA) (AASHTO M 154, ASTM C260/C260 M) are added during concrete mixing to create air bubbles in the fresh concrete that remain once the concrete has hardened. Most entrained air bubbles range in size from 10 to 100 μm (Kosmatka and Wilson 2011) and, ideally, are uniformly dispersed throughout the HCP. Figure 1 shows an example of an entrained
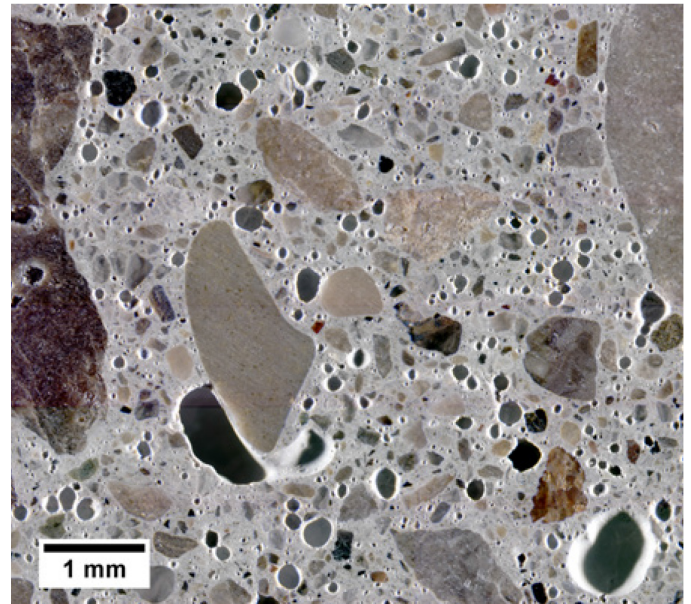

Figure 1. Stereo micrograph of entrained air void system (spherical bubbles) in hardened concrete. Larger, irregular voids are entrapped air. (source: Karl Peterson)

air-void system. Because the direct measurement of the size and distribution of the air voids in concrete is difficult and time-consuming, it is most common to specify the required total volume of air that will generally provide the needed bubble size and spacing. It is recognized that this is not necessarily adequate and thus alternate tests that make some assessment of the air-void system characteristics in fresh concrete, such as the air-void analyzer (AVA) and Super Air Meter (SAM), are discussed in the companion Tech Brief, *Ensuring Durability of Concrete Paving Mixtures-Part II: Test Methods*.

The required air content in ACI (2011) is dependent on both the exposure condition (presented in table 3) and the paste content (or mortar fraction) in the concrete (which is often related to the nominal maximum aggregate size, as shown in table 5). The more paste that is present, the more air is required to protect it. ACI (2008) mixture proportions are based on an assumption of angular coarse aggregates; thus, it is sometimes possible to reduce the required air by approximately 1 percent if rounded aggregates are used. Additional air may be required for heavy deicer use.

Table 5. Recommended air content for exposure classes and nominal maximum aggregate sizes (ACI 2014).

Nominal maximum aggregate size, mm (in.)	Exposure Class F1 Air Content, %[1]	Exposure Class F2 and F3 Air Content, %[1]
9.5 (3/8)	6	7.5
12.5 (1/2)	5.5	7
19 (3/4)	5	6
25 (1)	4.5	6
37.5 (1-1/2)	4.5	5.5
50 (2)	4	5
75 (3)	3.5	4.5

1 A reasonable field tolerance on air content is recommended as ±1-1/2% (ACI 2008).

In addition to total air content, the maximum water-to-cementitious-materials ratio (*w/cm*) is also typically specified. This is a recognition that the overall porosity of the HCP decreases as *w/cm* decreases, resulting in a decrease in permeability and an increase in strength. ACI (2011) recommends a maximum *w/cm* of 0.45 for a freeze-thaw Exposure Classes of F1, F2 and F3. Further, if the pavement is hand-finished, the supplementary cementitious materials (SCM) content is limited to a maximum of 25 percent fly ash or 50 percent slag cement, by mass of total cementitious materials. More recently, there has been recognition that formed and machine finished surfaces, such as slipformed pavement surfaces, are not greatly at risk of scaling and thus these limits are not applicable.

In summary, freeze-thaw damage in the HCP is mitigated primarily through the creation of an effective entrained air-void system in the concrete, in which the spherical air voids are spaced closely enough to relieve the stress generated through hydraulic and osmotic pressures. Current guidance suggests that the total volume of entrained air required to prevent damage is related to the overall volume of the mortar requiring protection and the freeze-thaw conditions to which the concrete is to be exposed. It is assumed that the total volume of air is related to the size and spacing of the entrained air voids; an assumption that is not always true. As freeze-thaw conditions become more severe, the maximum allowable *w/cm* is reduced, thereby lowering the concrete permeability and increasing the strength.

Deicer Scaling

As noted previously, the application of chemical deicers can amplify the physical mechanisms responsible for freeze-thaw deterioration of concrete. This can lead to surface scaling, as shown in figure 2. Concrete that will be subjected to deicers must be air-entrained and have a *w/cm* below 0.45 to prevent scaling due to the use of deicing chemicals. If the concrete is to be hand-finished, the total SCM content in the cementitious materials is typically limited to a maximum of 25 percent fly ash or 50 percent slag cement. Other guidance has suggested a minimum of 50 percent by mass of the total cementitious materials should be portland cement.

Figure 2. Concrete surface suffering deicer scaling. (source: Tom Van Dam)

Although these requirements address the inherent durability of the concrete material, the potential for deicer scaling is significantly affected by what happens during and immediately following construction. For maximum resistance to deicer scaling, finishing activities must be kept to a minimum and good curing practices must be followed. This is discussed in more detail in the companion Tech Brief, *Ensuring Durability of Concrete Paving Mixtures-Part II: Test Methods*.

Chemical Deicer Attack

Chemical deicer attack is a new area of concern for pavements located in areas where the use of calcium chloride or magnesium chloride is becoming more common (Sutter et al. 2006; Jones et al. 2013). An understanding of the mechanisms responsible for this form of attack is currently emerging and the true extent of risk to concrete pavement performance is still being studied. Nevertheless, many consider it prudent to consider the potential for chemical deicer attack when designing concrete pavements that will be subjected to these types of deicing chemicals.

The primary mechanism thought to be responsible for chemical deicer attack is the formation of calcium oxychloride, which involves the calcium hydroxide present in most HCPs in the deleterious chemical reaction (Sutter et al. 2006; Jones et al. 2013; Weiss and Farnam 2015). To address this distress, current recommendations are to follow those above for freeze-thaw deterioration of HCP, and to use a pozzolanic SCM (i.e. fly ash, slag cement, or silica fume) in the concrete mixture to reduce the amount of calcium hydroxide available for chemical deicer attack. Research has suggested that the use of

penetrating silane or siloxane sealants, applied directly to concrete joints, may provide a barrier to the ingress of aggressive chemical deicers and offer some level of protection (Weiss and Farnam 2015).

Aggregate Freeze-Thaw Deterioration

Certain coarse aggregates either fracture or dilate when subjected to repeated cycles of freezing and thawing in a critically saturated state, resulting in cracking of the surrounding mortar and deterioration of concrete. The manifestation of this deterioration is known as D-cracking. Figure 3 shows an example of aggregate freeze-thaw damage. Often preceded by dark staining on the surface of the concrete pavement, cracking will eventually appear in an hourglass shape on the pavement surface at affected joints and cracks (Van Dam et al. 2002a; 2002b).
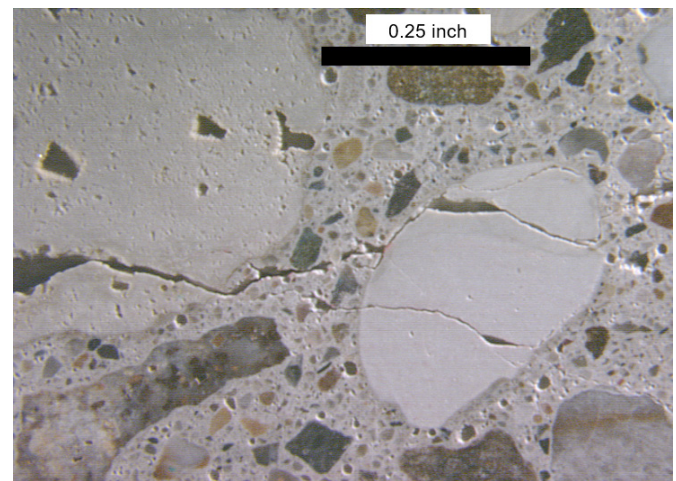

Figure 3. Stereo micrograph showing carbonate aggregate fractured due to freeze-thaw damage. (source: Karl Peterson)

Most D-cracking susceptible aggregates are of sedimentary origin, commonly composed of limestone, dolomite, or chert (Stark 1976), with limestone aggregates found in a band running from Kansas to Michigan being particularly susceptible (Taylor and Wang 2015). Susceptible aggregate generally have higher total porosity and a higher proportion of medium-sized pores (0.1 to 5 μm), which allows a significant volume of water in freezable pore space when saturated (Kosmatka and Wilson 2011).

The most effective means of preventing this distress is by prohibiting the use of susceptible aggregate by specifying that coarse aggregates pass certain freeze-thaw requirements. There are a number of test methods that have been employed to screen aggregates for susceptibility to freeze-thaw

deterioration, which the most common methods being regional variations of AASHTO T 161/ASTM C666. The acceptance criteria are set by each agency, with typical acceptance threshold values as follows:

- Durability factor (DF) should be greater than 60 to 80 at 300 to 350 cycles.

- Length change (dilation) should be less than 0.035 percent at 300 to 350 cycles.

Another screening test method is the Iowa Pore Index test, which has been used by the Iowa DOT and others. The acceptance criteria set by the Iowa DOT are based on aggregate Durability Classes defined in Section 4115 of the IADOT *Standard Specifications for Highway and Bridge Construction* for interstate pavements, which requires a Secondary Pore Index of 20 or less (Iowa DOT 2012).

Some states have found that reducing the maximum size of the susceptible coarse aggregate has been effective in reducing freeze-thaw deterioration of aggregate in many instances. In such cases, it may be necessary to add in larger-sized non-susceptible coarse aggregates to maintain a suitable aggregate blend to maintain a relatively low HCP volume.

Alkali-Silica Reactivity (ASR)

ASR results in the formation of an expansive gel that fractures the affected aggregate particles and surrounding paste, an example of which is shown in figure 4. Extensive information is available regarding the mechanisms responsible for ASR and the strategies available to mitigate it (Thomas, Fournier, and Folliard 2013). For highway applications, AASHTO PP 65 (*Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction*) provides the most comprehensive recommendations on mitigating ASR. The protocols are detailed and should be consulted if ASR is a concern.

Figure 4. Stereo micrograph showing fine aggregate particles fractured due to ASR. Sodium cobaltinitrite was used to stain the ASR gel yellow, making it readily visible in the affected aggregate particles, cracks, and in the cement paste. (source: Karl Peterson)

There are basically four strategies available to mitigate ASR (Thomas, Fournier, and Folliard 2013):

1. **Avoid the use of reactive aggregate.** Aggregate reactivity must be assessed through competent and thorough testing, including a combination of petrographic analysis (ASTM C295), expansion testing of mortar (AASHTO T 303/ASTM C1260) or concrete (ASTM C1293), and an evaluation of field performance. It is emphasized that this option may not always be the best, as non-reactive aggregates might not be locally available, local reactive aggregates with otherwise suitable properties for use in concrete are available at low cost, or because of uncertainty in the test results.

2. **Minimize the total alkalis in the concrete mixture, considering both the alkalinity and amounts of the cementitious materials contained in the mixture.** The required limit will vary with a number of factors, but a maximum alkali limit of 3.0 lbs/yd^3 sodium oxide equivalent (Na$_2$O$_{eq}$) has been adopted in the AASHTO PP 65 protocol for a high level of prevention.

3. **Use SCMs.** This is a very efficient strategy to control ASR, and a number of test methods (AASHTO T 303/ASTM C1567) are available to assess the effectiveness of SCMs at different dosage rates. Table 6 provides general recommendations regarding typical SCM levels.

4. **Use lithium-based admixtures**. Lithium compounds, particularly lithium nitrate, have demonstrated effectiveness in mitigating ASR.

The effectiveness is highly dependent upon aggregate reactivity; thus, testing must be done to determine the required dosage of lithium-based admixture.

Table 6. Required levels of SCMs to control ASR (from Thomas, Fournier, and Folliard 2013).

Type of SCM	Level required (%)
Low-calcium fly ash (<8% CaO; typically Class F fly ash)	20 to 30
Moderate-calcium fly ash (8 - 20% CaO ; can be Class F or Class C fly ash)	25 to 35
High-calcium fly ash (>20% CaO; typically Class C fly ash)	40 to 60
Silica Fume	8 to 15
Slag Cement	35 to 65
Metakaolin (calcined kaolin clay)	10 to 20

These four strategies are presented in AASHTO PP 65, in which aggregate reactivity for ASR is judged as follows:

- Consideration of field performance, taking into account any differences in materials and mixture design that may have occurred.

- Petrographic analysis of the aggregate (ASTM C295) to determine the presence of potentially reactive minerals.

- Mortar bar expansion (AASHTO T 303/ASTM C1260) not greater than 0.10 percent after 14 days immersion in 1 M NaOH solution at 176 ºF, and expansion of concrete prisms (ASTM C1293) of not greater than 0.040 percent at 1 year.

If the aggregate is identified as being potentially ASR reactive, AASHTO PP 65 requires that it be rejected or used with appropriate preventive measures as discussed above using either a prescriptive approach or performance approach.

The prescriptive approach in AASHTO PP 65 uses the accelerated mortar bar test (AASHTO T 303/ASTM C1260) to classify the reactivity of the aggregate from non-reactive to very highly reactive. From these results, a level of ASR risk is defined considering the exposure conditions. The level of prevention required is then determined based this

level of ASR risk and the classification of the structure based on the consequences of having an ASR problem. Pavements are commonly considered as being Class S3, meaning that the consequence of minor ASR damage can cause significant safety, economic, or environmental consequences, yet minor risk of ASR is acceptable. The final step within the prescriptive approach to mitigation in the AASHTO PP 65 is to identify preventive measures based on the level of prevention required. Options including limiting the alkali content of the cementitious materials, using SCMs, or a combination of the two, as discussed previously.

The performance approach in AASHTO PP 65 recommends that the concrete prism test (ASTM C1293) be used to evaluate the efficacy of ASR mitigation strategies such as SCMs or lithium compounds. As the test will require 2 years to conduct (requiring that expansion of concrete not exceed 0.040 percent at 2 years), it is recommended that a range of SCM or lithium types and dosages be tested to ensure an effective strategy can be determined. A method is also provided for the use the accelerated mortar bar test (AASHTO T 303/ASTM C1567) to determine SCM or lithium dosage once the longer duration concrete prism test has been conducted for the specific aggregates under consideration.

The detailed approach espoused in AASHTO PP 65 should be consulted if either ASR or ACR conditions potentially exist.

Sulfate Attack

Sulfate attack is not thought to be a common problem for pavements, although it is a concern for concrete placed in contact with soil in many areas of the western U.S. If the soil is tested and the sulfate exposure class is S1 through S3 as described in table 4, mitigation is required.

Mitigation strategies focus on reducing concrete permeability (decrease the *w/cm*, use acceptable SCMs) and using cementitious systems that are resistant to sulfate attack, for example, AASHTO M 85 Type II or V portland cement, AASHTO M 295 moderate or high sulfate-resistant blended cements, with the addition of pozzolanic SCMs or slag cement (ACI 2008; ACI 2014; Kosmatka and Wilson 2011). ACI (2011) prescriptive recommendations for mitigation strategies to mitigate sulfate attack are shown in table 7, while table 8 provides ACI (2008) performance requirements.

Table 7. Prescriptive requirements to protect concrete against damage by sulfate attack from external sources of sulfate (ACI 2008, ACI 2014).

Severity of Potential Exposure	w/cm by mass, max	ASTM C150 Cement Type**	ASTM C595 Cement Type**	ASTM C1157 Cement Type**
S0	No w/cm restriction	No special requirements	No special requirements	No special requirements
S1	0.50 §	Type II*†	IP (MS), IS (<70) (MS), IT(P<S<70)(MS), or IT(P≥S)(MS)	MS
S2	0.45§	Type V‡	IP (HS), IS (<70)(HS), IT(P<S<70)(HS), or IT(P≥S)(HS)	HS
S3	0.40§	Type V plus pozzolan or slag cement**	IP (HS), IS (<70)(HS), IT(P<S<70)(HS), or IT(P≥S)(HS)	HS***

*An ASTM C150 Type III cement with the optional limit of 8% C3A may be permitted.
† For seawater exposure, other ASTM C150 Cement Types with C3A contents up to 10% are permitted if w/cm does not exceed 0.40.
‡ An ASTM C150 Type III cement with the optional limit of 5% or ASTM C150 Cement of any type having expansion at 14 days no greater than 0.040% when tested by ASTM C452 may be permitted.
§ Values applicable to normal weight concrete.
** Any source of pozzolan or slag cement added at a dosage that has been shown to improve the sulfate resistance of a portland cement.
*** For exposure class S3, ASTM C1157 HS cement must contain pozzolan or slag cement.

Table 8. Performance requirements to protect concrete against damage by sulfate attack from external sources of sulfate (ACI 2008).

Severity of Potential Exposure	w/cm by mass, max	Max Expansion At 6 Months†	Max Expansion At 12 Months†	Max Expansion At 18 Months†
S0	No w/cm restriction	-	-	-
S1	0.50 §	0.10 %	-	-
S2	0.45§	0.05%	0.10%¥	-
S3	0.40§	-	-	0.10%

† Maximum expansion when tested using ASTM C1012.
¥ The 12-month expansion limit can be used if the 6-month limit is not met but is not required if the 6-month limit is met.
§ Values applicable to normal weight concrete.

SUMMARY

Table 9 provides a general summary of mitigation strategies to address common MRD types that can affect concrete pavements.

Table 9. Summary of mitigation strategies to address common types of MRD in concrete pavement.

Type of MRD	Maximum *w/cm*	Cementitious Materials	Aggregate Properties	Other
Freeze-Thaw Deterioration of Hardened Cement Paste without Deicers	0.45	Limits exist for hand-finished slabs.	N/A	Add air-entraining admixture to establish protective air-void system.
Deicer Scaling/ Deterioration	0.45	Limits exist for hand-finished slabs.	N/A	In addition to entraining a protective air-void system, provide a minimum 30-day "drying" period after curing before allowing the use of deicers.
Chemical Deicer Attack	0.45	Use cementitious systems that produce relatively low levels of calcium hydroxide. Most often accomplished through the use of a low-calcium fly ash (i.e., Class F) or slag cement.	N/A	In addition to entraining a protective air-void system, some success has been reported in the use of penetrating, breathable sealants.
Freeze-Thaw Deterioration of Aggregate	N/A	N/A	Reject aggregate sources that do not pass AASHTO T 161 screening tests.	Crush susceptible aggregates to a size where they are no longer susceptible.
Alkali-Silica Reactivity (ASR)	N/A	Use AASHTO PP 65 to select low-alkali cement and SCMs to mitigate reactive aggregate.	Screen aggregates using AASHTO PP 65 to determine level of reactivity and to select a feasible mitigation strategy.	Mitigation can also include the use of lithium-based admixture.
External Sulfate Attack	Decrease *w/cm* with increasing level of exposure (see tables 7 and 8).	Select based on level of exposure (see tables 7 and 8). There are moderate-to-high sulfate-resistant cements. Class F fly ash and slag cement are effective at mitigating sulfate attack.	N/A	Performance specifications also exist based on ASTM C1012 testing up to 18 months.

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