

Tech Brief

MATERIALS-RELATED DISTRESS: AGGREGATES

Best Practices for Jointed Concrete Pavements

INTRODUCTION

Concrete is considered, in general, as a cemented-aggregate mixture. In other words, aggregates are central to concrete and are bound together by a cementitious matrix.

Aggregates comprise a significant volume in most concrete mixtures, which is generally desirable because aggregates are less expensive than components of the paste, they provide the load-carrying capacity of a concrete mixture, and they provide dimensional stability. However, some failure mechanisms in pavement concretes can be attributed to issues related to the quality of the aggregate.

The purpose of this tech brief is to provide pavement engineers with information to help them specify and choose materials that will last for the desired lifetime of the pavement. Paste-related mechanisms are discussed in a separate parallel tech brief.

ALKALAI-SILICA REACTION

What is it?

Alkali-silica reactivity (ASR) has been the subject of considerable research since first identified in the 1930s. This research has helped to improve the understanding of the reactions that occur, how to predict whether a given aggregate is prone to damage, and how to prevent this damage.

Broadly, the reaction is between siliceous minerals present in some aggregates (such as strained quartz, chert, trydimite, cristobalite, opal, or natural glass), water, and alkali hydroxides (normally from the cement). The reaction product, which is referred to as a gel, expands significantly in the presence of water.



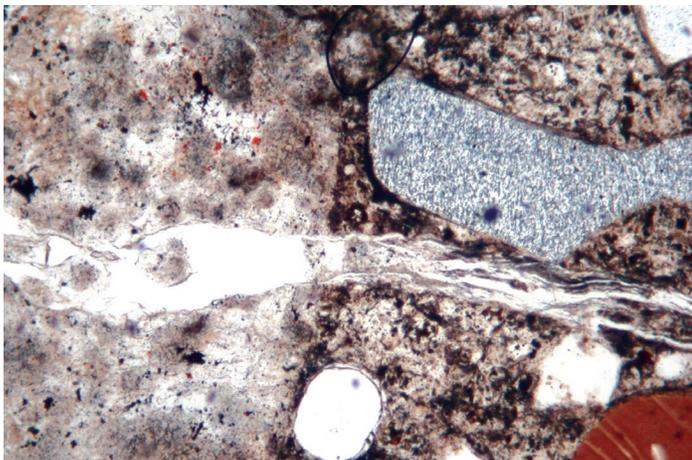
Thomas et al. 2011

Pavement cracking caused by alkali-silica reactions



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Portland Cement Association

Alkali-silica reaction on a micro scale

Cracking therefore starts within and near the surface of the reacting aggregate and extends outward into the paste as the aggregate particles react. It is common to observe a reaction zone, which is referred to as a reaction rim, around the outside of reacting particles. However, the presence of a reaction rim alone is not sufficient to identify ASR, as many aggregates, and particularly gravels, will have a reaction rim appearance simply due to natural weathering of the aggregate materials.

On a macro scale, the distress manifests as cracks, often parallel to the free edge of a slab or as “map cracking” across the whole section. In extreme cases, a white deposit (i.e., gel) may be seen exuding from the cracks. Again, caution should be exercised in assuming a white exudate is gel as reaction products from other reactions may have a similar appearance (e.g., efflorescence).

Testing

Testing for potential reactivity is a challenge because, under ambient conditions, the reactions may take 10 years or more to be noticeable. Accelerating the reaction by heat or by adding alkalis skews the outcome, making the tests less reliable.

Two standard tests are currently recommended. ASTM C 1293 (2008) involves making a concrete prism with added alkali and storing the sample above water at room temperature for a year. This is considered the most diagnostic test, although it may provide false positives, but more importantly false negatives, and it is quite lengthy.

The alternative, ASTM C 1260 (2014), uses mortar bars stored in alkali solution at 70°C for two weeks. The test is generally overly conservative and may reject materials that perform satisfactorily in the field, although some materials in the northwestern states have been shown to pass the test and still expand in the field.

AASHTO PP65 (2010) is a provisional protocol that provides guidelines in the form of a decision tree that uses all of the information available (previous performance, petrography, and results from the two standard ASTM tests) to assess the risk of reaction in a given aggregate and the effectiveness of mitigation approaches. A similar protocol was recently adopted by ASTM (ASTM C 1778).

Prevention

Several approaches can be taken to reduce the risk of future distress in new pavements. The first is to avoid using reactive aggregates, but this is not always possible in some locations. Reducing the alkali content of the cementitious system will help, but this may not guarantee adequate protection for some aggregate types.

Another alternative is to use a sufficient amount of supplementary cementitious materials (SCMs) in the mixture. An SCM is effective because it will combine with alkali hydroxides in the concrete and produce additional calcium silicate hydrates (C-S-H), thereby reducing the alkali hydroxides in the concrete, and the production of more C-S-H reduces permeability, thereby slowing the ingress of water.

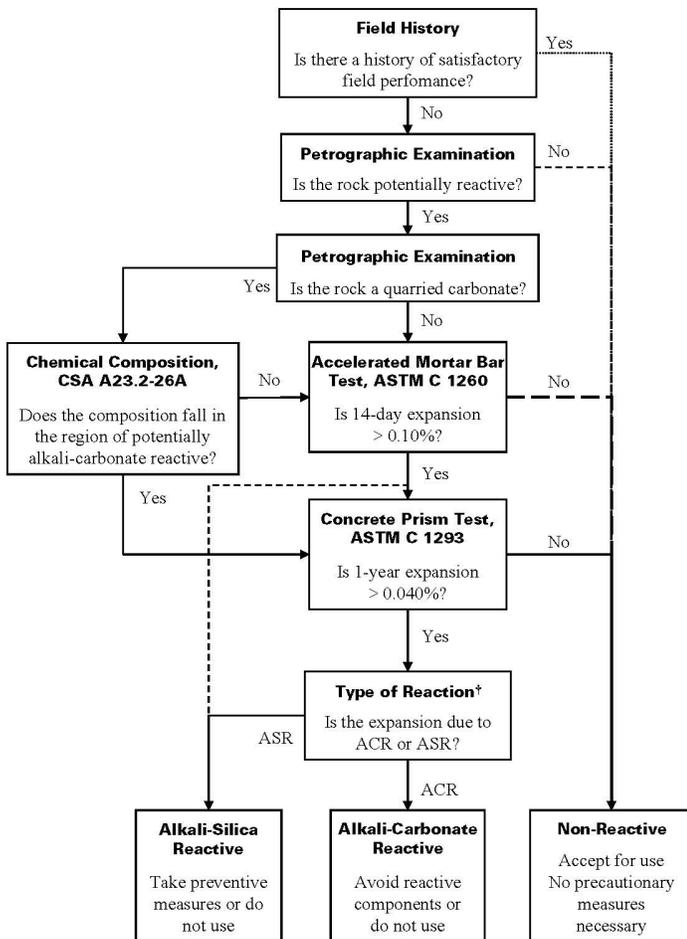
The lower the silica content of the SCM, the greater the amount of SCM needed to control expansions. In the case of fly ash, a low silica content is associated with a high calcium oxide content and therefore, PP65 and ASTM C 1778 prescribe fly ash in terms of the CaO content. Indeed, it is possible that an insufficient dosage of some SCMs may increase expansion until a pessimum requirement is exceeded.

The amount of a given product needed for a given aggregate should be selected by following the guidelines of AASHTO PP65 or ASTM C 1778. Care is needed because high SCM dosages may increase the risk of other problems, such as lower rates of strength development, early-age cracking, or scaling.



Thomas et al. 2011

Alkali-silica reaction cracks in a concrete pavement



Thomas et al. 2008

Decision tree specified by AASHTO PP65

Consideration may also be given to including lithium-based compounds (normally nitrate) in the mixture (FHWA 2002). These compounds have the effect of changing the nature of the reaction gel to a form that is less expansive in the presence of water.

Repair

Repair of existing pavements that are exhibiting ASR will depend on the extent of the reaction and the effects that it is having on serviceability. Initial effects will be that joints will close up, increasing the probability of blow-ups in hot weather, and that side drains and adjacent slabs may be moved. Severe cracking will reduce the ride quality and increase slab movement. A system that is cracking due to ASR may also be at a greater risk of other deterioration mechanisms, such as accelerated freezing and thawing.

Typically, action needs to be taken well before the reaction has run its course, meaning that any repair approach must be able to accommodate future expansions (FHWA 2002). Options include the following:

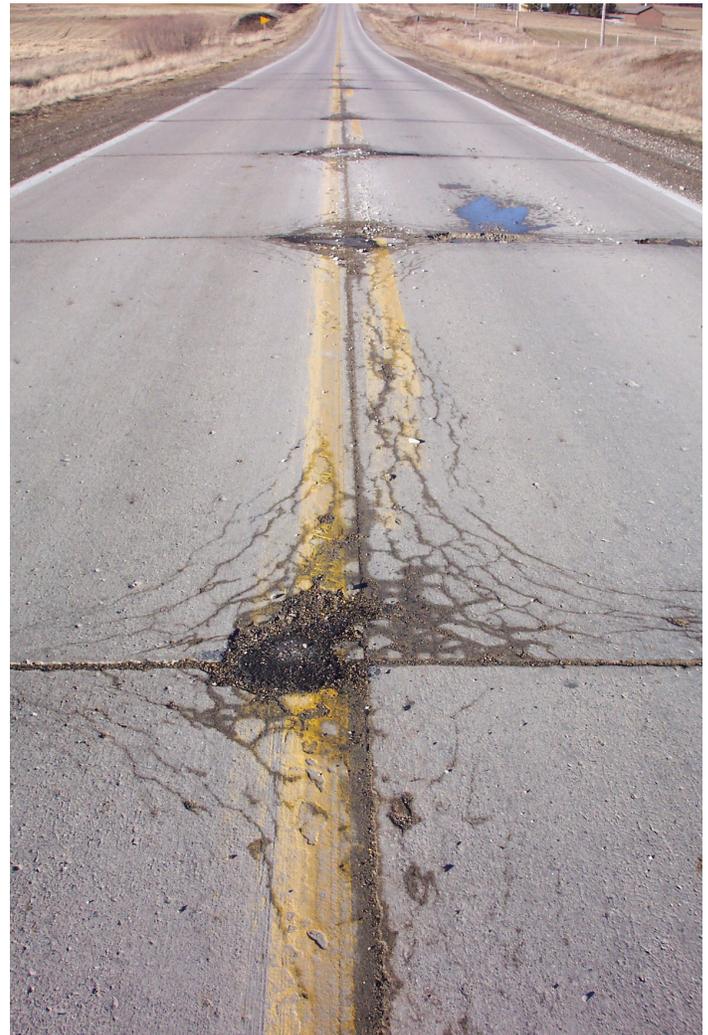
- Remove and replace the slab
- Rubblize the slab and place a new surface system
- Place an unbonded concrete overlay

The choice among these options depends on the severity of distress, local economics, and the desired service life of the repair.

D-CRACKING

What is it?

Portland cement concrete pavements in the midwestern states can suffer premature deterioration due to so-called D-cracking, a progressive distress associated with the use of coarse limestone aggregates that, when critically saturated, physically break down under repeated freeze and thaw cycles. The susceptible materials appear to be found in a band running northwest from Kansas up to Michigan.



Jim Grove

D-cracking on a pavement surface

D-cracking starts when water enters the aggregate through its pore structure, which could occur over an extended period of time (i.e., non-winter seasons). When temperatures drop and a freezing front moves through the concrete, water may be expelled from the aggregate by the expansion of ice formation. If the pore structure of the aggregate is such that water cannot exit quickly, the absorbed water subsequently freezes causing expansion and cracking in the aggregate and the surrounding mortar.

Normally, water in pores that are smaller than 0.1 μm does not freeze because of the very high specific surface area of the smaller voids. Aggregates with small or discontinuous pores also tend to be durable because, initially, the water has limited access into them.

Aggregates with large pores, i.e., greater than 0.2 μm , are also durable because freezing water can be expelled easily without causing deterioration to the aggregate.

The intermediate pores, with a size ranging from 0.04–0.20 μm , are the least durable because surface tension between the water and the pore surfaces limits the movement of the water out of the aggregate, thus increasing the risk that the system will become critically saturated (Pigeon and Pleau 1995, Marks and Dubberke 1982).

Some aggregates may also contain clay materials or other minerals that expand in the presence of water and some deicing salts, exacerbating the distress.

D-cracking is observed in pavements and slabs on grade. The cracks normally begin in the lower portion of the slab and move upward as a result of groundwater being wicked into the concrete from below and through the transverse and longitudinal joints.

D-cracking almost always appears first along joints and works toward the slab center. It may be observed that black, blue, white, and gray deposits of calcium carbonate and dirt fill the cracks. If cores are extracted, cracking is observed to start in the aggregates and normally runs parallel to free surfaces.

Testing

Each state with aggregates prone to this type of deterioration tends to have its own approach to evaluation.

The Iowa Pore Index test was developed to relate freeze-thaw durability with the relative size and abundance of capillary pores in the aggregate (Marks and Dubberke 1982, Myers and Dubberke 1980). The test involves measuring the volume of water forced into a sample under high pressure over a period of 15 minutes.

This test is the basis of Iowa acceptance protocols, but it has not found broad acceptance elsewhere. The Iowa protocol also uses elemental analysis to quantify clay content and X-ray diffraction (XRD) analysis to characterize the mineralogy of the aggregate system with respect to its position on the limestone/dolomite spectrum.

Other tests include a pressure release approach and freeze-thaw testing of a standardized mixture (ASTM C 1646 2008). Some authorities refer to the magnesium soundness test, but this is not considered a reliable approach.

Most agencies use an approach based on historical performance and ledge inspection at the quarry. Some agencies also limit the use of limestones altogether. A challenge to any acceptance process is to ensure that the aggregate in any given load of concrete is from an acceptable source.

Prevention

The only way to totally prevent distress is to avoid using at-risk aggregates. Limiting the maximum size can reduce the rate of distress, but failure is still possible, albeit after considerable service (e.g., 40 years). In some cases, such as glacial gravels that contain a mixture of different mineralogical sources, a certain amount of expansive aggregate may be acceptable, but data are still being collected to quantify this limit.

Repair

Repair of existing pavements that are exhibiting D-cracking, like those undergoing ASR, must be able to accommodate future expansions. Options include the following:

- Remove and replace the slab
- Rubblize the slab and place a new surface system
- Place an unbonded concrete overlay

It is not uncommon for an asphalt overlay to be applied, but this is typically a one-season repair because the damaged concrete will continue to expand on freezing and shrink on thawing, leading to large vertical movements at the joints.

OTHER CONTAMINANTS

Table 4 of ASTM C 33 (2013) provides limits on a number of aggregate materials and properties in an effort to reduce the potential for premature failure of concrete:

- Clay lumps and friable particles are controlled because excess amounts will significantly increase the water demand of the mixture due to their high specific surface area and exacerbate expansion in the presence of water. They may also tend to reduce flexural strengths.

- Low-density chert is limited because of its propensity to cause popouts at the surface of slabs on grade or react in ASR (NRMCA 2007).
- Material finer than 75 μm is also limited because of its risk of containing clays. This limit may be relaxed for crushed aggregates because there is greater assurance that the fine material is not clay.
- Coal and lignite are undesirable because of their effect on water demand and the risk of popouts and staining.
- Low-abrasion-resistant materials need to be controlled in applications that are subjected to polishing (e.g., hard wheels) and abrasion (e.g., studded tires, chains).
- Magnesium sulfate soundness is used to indicate an aggregate's ability to resist the effects of wetting and drying and freezing and thawing cycles.

ASTM C 33 imposes different limits based on the sort of structure or element in which the concrete is going to be used.

COEFFICIENT OF THERMAL EXPANSION

What is it?

Coefficient of thermal expansion (CTE) is defined as the change in unit length (or amount that concrete changes dimensionally) per degree of temperature change.

Typical CTE ranges for aggregates

	Coefficient of Thermal Expansion	
	10-6/ $^{\circ}\text{C}$	10-6/ $^{\circ}\text{F}$
Granite	7-9	4-5
Basalt	6-8	3.3-4.4
Limestone	6	3.3
Dolomite	7-10	4-5.5
Sandstone	11-12	6.1-6.7
Quartzite	11-13	6.1-7.2
Marble	4-7	2.2-4

Data Source: FHWA 2011

CTE is a critical input in the mechanical/empirical design procedure because daily and seasonal temperature-based movements can have an effect on the curling of the system. In addition, differential movements between an overlay and the original layer can exacerbate curling and increase cracking risk. The property is typically not measured, but, because it does have a strong influence on thickness design, it should be recorded at the mixture design stage.

This concrete property is strongly influenced by the aggregate type, with limestone-based aggregates exhibiting lower expansion ranges than siliceous materials.



Xuhao Wang/National CP Tech Center

Popout on a concrete surface

Testing

A standard test method for determining the CTE of concrete has been adopted by AASHTO as TP60-00 (2005). The method involves measuring the length change of a cylindrical saturated concrete specimen over a specified temperature range (10 $^{\circ}\text{C}$ to 50 $^{\circ}\text{C}$ or 50 $^{\circ}\text{F}$ to 120 $^{\circ}\text{F}$).

$$\text{CTE} = (\Delta L/L_0) / \Delta T$$

where,

ΔL = length change of the specimen

L_0 = initial measured length of the specimen

ΔT = temperature change

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