

Overview of Alkali-Silica Reactivity in Concrete Pavements



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Introduction

Alkali-aggregate reaction (AAR) is a potential distress mechanism for portland cement concrete (PCC) that results from a reaction between alkali hydroxides in the PCC pore solution and reactive aggregates. The most common occurrence of AAR is the alkali-silica reaction (ASR) that results from reactions involving specific siliceous aggregates. The much less common occurrence of AAR is alkali-carbonate reactivity (ACR), which occurs with very specific types of carbonate aggregates. Both forms of AAR generally manifest as map cracking over a large area accompanied with expansion-related distresses (e.g., joint closure, spalling, blowups). Methods for prevention or mitigation of ASR start with avoiding the use of reactive aggregates when possible. When reactive aggregates must be used, ASR can be mitigated in most cases by including in the concrete mixture supplementary cementitious materials (SCMs) or lithium compounds, limiting the alkali loading in the concrete, and minimizing exposure of the concrete to moisture. In the case of ACR, the distress cannot be mitigated and the only recourse is avoiding the use of the reactive aggregate.

Strategies for ASR mitigation, historically, have been developed on a state-by-state basis, with the most common approach being to limit the alkali content of the portland cement. Based on a recent review of all U.S. state highway agency (SHA) concrete specifications, 28 states impose a limit on the alkali content of portland cement, with 0.60 percent $\text{Na}_2\text{O}_{\text{eq}}$ maximum being specified as the limit in most cases (TMRC 2019).

Although in some cases there are similarities when comparing state specifications, overall there is no consistent approach adopted by

the states when it comes to ASR mitigation. To develop best practices, the Safe Accountable Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU), which was passed in 2006, provided funding for “refining and deploying proven technologies and methods to address ASR” (Federal Highway Administration 2013). To accomplish this task, the Federal Highway Administration (FHWA) established the ASR Development and Deployment (D&D) Program and conducted a comprehensive research program leading to development of multiple publications¹ that culminated as AASHTO PP 65-11 Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction. This provisional guide was published as a full standard AASHTO R 80-17 in 2017. In parallel, based on the same research, ASTM International published ASTM C1778-20 Standard Guide for Reducing the Risk of Deleterious Alkali-Aggregate Reaction in Concrete in 2014.

These two guide documents are essentially equivalent and provide both performance and prescriptive approaches to AAR prevention and mitigation and are widely acknowledged as being the best available knowledge on AAR. Yet to date, with few exceptions, SHAs have not adopted the guidance in these documents either in whole or in part. The purpose of this brief is to introduce these guides and provide background to assist SHAs with implementation of this state-of-the-art information. Adopting a common approach to AAR across all states, and particularly to mitigating ASR, will benefit all stakeholders in the highway infrastructure construction industry and lead to more durable and longer lasting structures.

Background

ASR Reaction Mechanism

ASR is a concrete distress resulting from a chemical reaction between alkali hydroxides dissolved in the concrete pore solution (i.e., NaOH and KOH) and reactive siliceous components of susceptible aggregates (Thomas et al. 2013). The product of the reaction is a gel material that significantly expands in the presence of moisture, destroying the integrity of the weakened aggregate particles and the surrounding cement paste (Thomas et al. 2013). As the gel expands, it can exert potentially damaging tensile pressures from within the concrete's cementitious paste matrix. These stresses may be large enough to cause bond and shear failures between concrete and reinforcement, and widespread “map cracking” as shown in Figure 1.

For ASR to occur, three components are required: 1) source of alkalis, 2) reactive aggregate, and 3) water (Thomas et al. 2013). The primary source of alkalis is the portland cement, which releases calcium and alkali ions as it hydrates (i.e., reacts with water). Although ASR is associated with the alkalis, it is actually hydroxyl ions (-OH) in the pore solution that attack the aggregate (Thomas et al. 2013). The attack is attributed to the alkalis because as the alkali ion concentration in the pore solution increases, there is a resulting increase in hydroxyl ion concentration (i.e., increased pH). With time, the hydroxyl ions in the concrete pore solution may react with certain forms of silica contained in the aggregate to form the alkali-silica gel product. Figure 2 shows a micrograph from a petrographic thin section where a reactive aggregate, cracking, and silica gel are highlighted using petrographic microscopy.

What makes some forms of silica reactive, and some not, is the crystalline structure of the silicate mineral being attacked. Minerals that are highly-ordered, or crystalline (i.e., pure quartz), have strong bonds between the silicon and oxygen atoms and are capable of resisting degradation due to hydroxyl ions, at normal temperature and pressure. When the atoms of a material are less ordered, or have no order such as an amorphous material (e.g., glass), the bonds between atoms are weaker and more readily broken down by hydroxyl ions. In other cases, the mineral structure of the aggregate may be micro-crystalline (e.g., cryptocrystalline quartz, chalcedony, or chert). These minerals have very small crystals that provide only short-range order in the mineral structure and as a result they react more like a glass (Thomas et al. 2013).

Minerals are the building blocks of rocks, and rocks are what we process to use as aggregate. Therefore, it is important to think of reactive rocks, rather than reactive minerals, although it may be only one mineral constituent in the rock that is reactive, and possibly only a small percentage of the rock by volume. Examples of rocks that are known to



Figure 1. An example of ASR-related map-cracking in concrete pavement (Photo Credit L. Sutter)

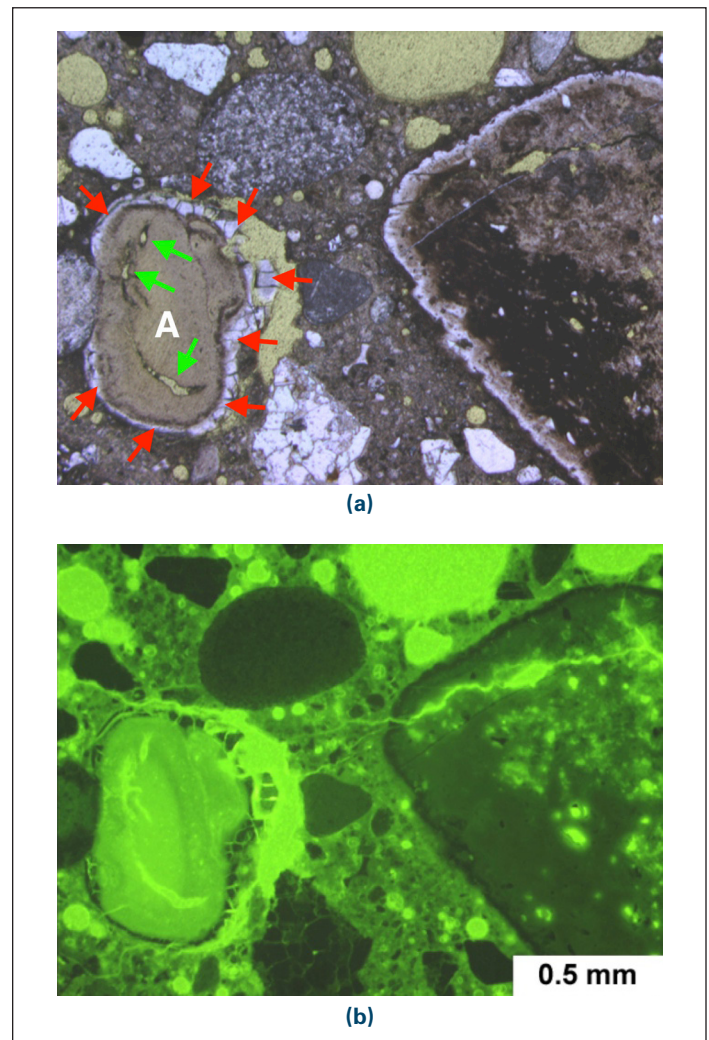


Figure 2. Micrograph of a petrographic thin section shown in plane polarized light (a) and epifluorescence (b). Aggregate (A) has reacted and expanded causing cracks to propagate through the paste and adjacent aggregates, which are highlighted using epifluorescent microscopy (b). Green arrows in (a) highlight an internal crack from aggregate expansion, red arrows highlight silica gel deposits around the aggregate, yellow regions in (a) are epoxy filled cracks and voids, which are further highlighted in epifluorescence (b). Bright green areas in (b) are cracks and voids, dark green areas are aggregates. (Micrographs courtesy of Karl Peterson, University of Toronto)

be potentially ASR reactive include rhyolite, dacite, latite, andesite, tuffs, shale, slate, sandstone, siltstone, quartzite, granites, grano-diorites, and granite gneisses (Thomas et al. 2013). This list is by no means complete, but it shows that a large number of rocks may be ASR reactive and deleterious to concrete.

As the gel product forms in the presence of water, the gel will imbibe water out of the surrounding cement paste and swell (Farny and Kerkhoff 2007). It has been found that an internal relative humidity of at least 75 to 80 percent is needed for deleterious ASR to occur (Farny and Kerkhoff 2007, Thomas et al. 2013). This internal relative humidity is common in transportation structures.

The concentration of alkali in the pore solution, or alkali loading, is a key factor affecting the rate of ASR development. Alkali loading is calculated based in part on the alkali content of the cement but is also affected by the total quantity of cement used in the concrete. The alkali content of cement is expressed as equivalent alkalies ($\text{Na}_2\text{O}_{\text{eq}}$) where $\text{Na}_2\text{O}_{\text{eq}} = (\text{wt. \% Na}_2\text{O}) + (0.658 \times \text{wt. \% K}_2\text{O})$. Alkali loading is given by the product of $\text{Na}_2\text{O}_{\text{eq}}$ and the cement content, expressed per unit volume of concrete. For example, if the concrete mixture is using 517 lb/yd³ [307 kg/m³], and the cement has a $\text{Na}_2\text{O}_{\text{eq}}$ value of 0.77 percent, then $517 \times 0.0077 = 4.0$ [307 \times 0.0077] = 2.36, meaning the alkali loading of the mixture is 4.0 lb/yd³ [2.36 kg/m³]. It is very common for SHAs to limit the equivalent alkali content of the portland cement, but this does not account for the cement content, which clearly affects the alkali loading of the mixture. To illustrate, Table 1 provides results of calculations for two different alkali contents and two different portland cement contents.

Historically, ASTM and AASHTO portland cement specifications had an optional requirement for the equivalent alkali content that limited $\text{Na}_2\text{O}_{\text{eq}}$ to 0.60 percent or less. This became known as “low-alkali cement” and many SHAs adopted this, often as their only provision to protect the concrete from deleteriously reactive aggregates. In 2019, recognizing that alkali loading was much more important, ASTM and AASHTO removed this optional requirement from their portland cement specifications and replaced it with the mandatory requirement to report the equivalent alkali content on mill test reports for all portland cements. They also added a note directing the specifier to their respective AAR guide for information on determining alkali loading. However, 0.60

percent $\text{Na}_2\text{O}_{\text{eq}}$ is still a very common SHA cement specification limit and often it is still the only action specified to prevent ASR. It is important to realize that depending on the reactivity of the aggregate and the portland cement content of the concrete mixture, the 0.60 percent $\text{Na}_2\text{O}_{\text{eq}}$ limit may not be sufficient to mitigate ASR.

Alkali is a component of the cement but alkali can also come from other sources, both internal and external. Internal sources of alkalies, other than the cement, include supplementary cementitious materials, chemical admixtures, aggregates, and even the mixture water. As will be discussed, alkalies from materials other than portland cement are taken into consideration in the guides, but they are not included in determining the alkali loading. External sources of alkalies include deicing salts, seawater, groundwater, and water from industrial processes that permeate the concrete. These cannot be completely predicted in advance and their presence could exacerbate AAR.

ACR Reaction Mechanism

Alkali carbonate reaction occurs with carbonate aggregates that have a very specific mineralogical texture (Thomas et al. 2013). The known ACR reactive aggregates are specific dolomitic limestones (i.e., dolomite is chemically $\text{CaMg}(\text{CO}_3)_2$), which is a limestone (CaCO_3) containing dolomite rhombohedra, clay, and silica suspended in a limestone matrix. Most dolomitic limestones are not ACR reactive and more generally, most carbonate rocks are not ACR reactive.

In ACR reactive aggregates, the dolomite rhombohedra undergo a reaction called dedolomitization where the dolomite reacts with alkali to form calcite, brucite and most importantly alkali carbonate (Thomas et al. 2013). The exact mechanism of ACR expansion is not clear. Theories include expansion due to swelling of the clay or growth due to the products of dedolomitization (Thomas et al. 2013). The key point of the reaction is that as the aggregate reacts to form alkali carbonate, that alkali carbonate reacts with calcium hydroxide in the cement paste and regenerates hydroxyl ions to continue the reaction. In other words, the reaction is self-sustaining and therefore cannot be mitigated (Thomas et al. 2013).

It should be pointed out that carbonate aggregates containing reactive silica can be ASR reactive. The most classic example is Spratt limestone which is used widely as a test aggregate

Table 1. Example calculations of alkali loading for two different alkali contents and two different portland cement contents

Cement Alkali Content $\text{Na}_2\text{O}_{\text{eq}}$, %	Cement Content lb/yd ³ [kg/m ³]	Concrete Alkali Loading lb/yd ³ [kg/m ³]
0.50	500 [297]	2.5 [1.5]
0.50	700 [415]	3.5 [2.1]
0.70	500 [297]	3.5 [2.1]
0.70	700 [415]	4.9 [2.9]

for ASR mitigation experiments. Carbonate aggregates may contain chert or other reactive forms of silica and SHA aggregate specifications routinely screen for these types of rocks. In most cases, it is possible to mitigate ASR when it occurs with carbonate aggregates.

ASR Mitigation Mechanisms

Clearly, the most effective way to avoid ASR is to avoid the use of reactive aggregates. Non-reactive aggregates are, however, limited in availability in some geographic areas and the cost of transportation precludes using aggregate sources that are not local. When using a reactive aggregate is necessary, steps must be taken to mitigate deleterious ASR. The most common approach is to use SCMs in the concrete mixture, which is a proven approach and effective in most instances. The most widely used SCM is low calcium fly ash (i.e., $\text{CaO} \leq 18\%$ by mass) but slag cement is also used as are silica fume, natural pozzolans and low alkali ground glass pozzolan (i.e., $\text{Na}_2\text{O}_{\text{eq}} \leq 4.5\%$ by mass).

The specific mechanism by which an SCM mitigates varies with the material. Most SCMs are pozzolans or have pozzolanic phases; pozzolanic materials react with hydroxyl ions, much like an ASR reactive aggregate does. The reaction produces calcium silicate hydrate (C-S-H) phases similar to the reaction product that forms during the hydration of portland cement, or in some cases calcium aluminate hydrate phases. In both cases, the result is hydroxyl ions are bound in the reaction products.

Slag cement is not pozzolanic but reacts to form hydration products that also bind hydroxyl ions. The important contribution of an SCM is to reduce the hydroxyl ion concentration in the pore solution by binding hydroxyl ions in the products of hydration. Forming a hydration reaction product with a low calcium to silica ratio increases the binding of hydroxyl in the reaction product. Therefore, the silica content of the SCM is the important characteristic that effects hydroxyl binding and effectiveness at mitigating ASR decreases as the silica content decreases (Thomas et al. 2013).

Ranking SCMs, silica fume is the most effective followed by metakaolin, low-calcium fly ash, and slag cement. Other materials such as high-calcium fly ash (i.e., $\text{CaO} > 18\%$ by mass) can often be used at relatively high replacement levels as compared to low-calcium fly ash. The alkali content of the SCM is also important and high-alkali fly ash with $\text{Na}_2\text{O}_{\text{eq}} > 5.0\%$ by mass is less effective at mitigating ASR (Thomas et al. 2012).

Another approach to mitigating ASR is to use lithium nitrate in the concrete mixture. The exact mitigation mechanism is unknown but the efficacy has been demonstrated (Thomas et al. 2007). Regarding the mitigation mechanism, one common theory is that lithium salts react with reactive silica, just like other alkali salts, but the reaction product is an insoluble

lithium-silicate that does not imbibe water and therefore it does not swell. This lithium silicate will form around reactive aggregate particles and by doing so, it provides a barrier that protects the reactive silica from attack by hydroxyl ions (Thomas et al. 2013).

Testing for AAR

There are a number of tests for identifying ASR or ACR reactive aggregates, or for evaluating ASR mitigation strategies. For brevity, only those currently referenced in AASHTO R 80 and ASTM C1778 are discussed here.

Tests for Identifying ASR Reactive Aggregates

In the guides, two tests are used for assessing aggregate reactivity. The first is ASTM C1293, *Standard Test Method for Concrete Aggregates by Determination of Length Change of Concrete Due to Alkali-Silica Reaction*, also referred to as the Concrete Prism Test (CPT). Concrete prisms are cast using a single reactive aggregate (coarse or fine), and a single non-reactive aggregate (coarse or fine, opposite of the reactive aggregate). The aggregates are crushed if necessary and sieved to prepare prescribed gradations. The concrete is prepared using 708 lbs/cy (420 kg/m³) of portland cement having an alkali content of 0.8 – 1.0% $\text{Na}_2\text{O}_{\text{eq}}$. Additional alkali (NaOH) is added to the test mixture to attain a total alkali content of 1.25% by mass of portland cement, or the portland cement portion of a blended cement. The prisms are cast and cured for 24 hours, zero measurements performed, and then stored over water in sealed containers at 100°F (38°C). The test duration for evaluating aggregate is one (1) year, with expansion measurements performed periodically over that time. An aggregate with an expansion of 0.04% or less at the end of one year is typically interpreted as not requiring any ASR mitigation measures. Expansions above 0.04% indicate that some mitigation measures should be used.

The second test is ASTM C1260 (AASHTO T 303) *Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)*, which is also referred to as the Accelerated Mortar Bar Test (AMBT). To perform the test, mortar bars are prepared using a single aggregate that has been crushed and sieved to produce a specified gradation. The mortar bars are cured for 24 hours, removed from their molds, and an initial length measurement is made. The mortar bars are then placed in water in sealed containers and stored at 176°F (80°C) for 24 hours, after which they are removed and measured again for length. This measurement is taken as the zero reading. The bars are then transferred to sealed containers and immersed in a solution of 1N NaOH (sodium hydroxide) and stored at 176°F (80°C). Over the next 14 days the mortar bars are removed periodically, measured for length, and returned to the soak solution. The expansion is expressed as a percentage of the effective gauge length and based on the measured expansion, the aggregate is ranked as innocuous

(expansion less than 0.10%), potentially reactive (expansion of 0.10% to 0.20%), or reactive (expansion greater than 0.20%) (Thomas et al. 2013).

Comparison of ASR Tests

ASTM C1293 is generally considered the best measure of aggregate reactivity. The test is widely criticized for its long duration, but many SHAs do specify the test for screening aggregates. The test is also criticized for the leaching of alkalis from the prisms over time, potentially skewing the results. However, good correlation has been shown between ASTM C1293 results and field performance, both in real structures and in long-term exposure test blocks. As a result, ASTM C1293 is considered the best available test for assessing aggregate reactivity and as will be discussed, the best available test for assessing ASR mitigation strategies.

The AMBT test is favored in practice because of its relatively short duration of 16 days (2 days, preparation, 14 days to monitor expansion). However, the AMBT test does have drawbacks. The test conditions used in the AMBT are considered severe and the test has, in some cases, indicated an aggregate as being reactive when based on other testing or field performance it was shown to not be reactive when used in the field (i.e., false positive) (Thomas et al. 1997) Conversely, aggregates have passed the test and in subsequent testing or use they have exhibited unacceptable expansion (i.e., false negative) (Folliard et al. 2006).

The level of agreement between the CPT and AMBT was examined in one study where testing was performed in three laboratories using 39 different aggregates and a total of 182 material combinations. For all mixtures, the AMBT and the CPT were both performed and the results are summarized in Table 2. It can be seen that in approximately 77% of the cases, both tests agreed. In approximately 16% of the cases the AMBT test found an aggregate to be reactive while ASTM C1293 showed the same aggregate to be non-reactive (i.e., false positive).

Conversely, in about 7% of the cases, the AMBT test found an aggregate to be non-reactive when ASTM C1293 testing rated it as reactive (i.e., false negative) (PCA 2009). Because of the chance of rejecting an otherwise good aggregate, it is

Table 2. The level of agreement between the one-year CPT and 14-day AMBT conducted in three laboratories using 39 different aggregates and a total of 182 material combinations (PCA 2009)

		ASTM C1260	
		Passed	Failed
ASTM C1293	Passed	62 (34%)	29 (16%)
	Failed	13 (7%)	78 (43%)

recommended to not reject aggregates based solely on results from the AMBT; aggregates failing this test should be confirmed using the CPT (Thomas et al. 2008).

Correlation of ASR Tests

The CPT is the best available test for evaluating aggregates, but it does require one (1) year to complete the test. Because of this long duration, the AMBT is what most users want to rely on given the much shorter duration of that test. Knowing there are cases where the AMBT and CPT tests disagree, the guides require that correlation between the two tests be established before relying on the AMBT for assessing aggregates or mitigation strategies. This correlation requires that initially, an aggregate source must be tested using both tests and those results are then plotted on the graph shown in Figure 3. Referring to Figure 3, aggregates with combined test results plotting in Zone 2 can be safely evaluated using the AMBT. The reactivity of aggregates with expansions plotting in Zone 1 will be overestimated by the AMBT while the reactivity of aggregates plotting in Zone 3 will be underestimated. Both guides recommend the correlation between ASTM C1260 (AASHTO T 303) and ASTM C1293 be re-established every two years or sooner if petrography or other data suggests a change in the aggregate source may have occurred.

Modifications to ASR Tests

ASTM C1260 (AASHTO T 303) and ASTM C1293 provide the most reliable results when they are applied without modification. However the existing tests, particularly the

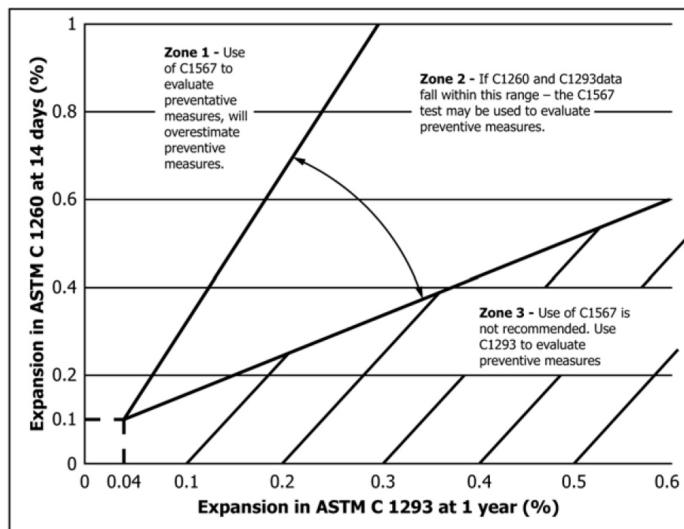


Figure 3. Plot of AMBT results versus CPT results for the same aggregates showing areas where the AMBT test overestimates expansion (Zone 1), underestimates expansion (Zone 3), and has reasonable correlation with ASTM C1293 (Zone 2). Reprinted, with permission, from ASTM C1778–20, *Standard Guide for Reducing the Risk of Deleterious Alkali-Aggregate Reaction in Concrete*, copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM, www.astm.org.

AMBT, are modified by some specifiers and such modifications are not supported by the guide documents.

One common AMBT modification is to specify more stringent expansion limits than those provided in the guides (e.g., reduce the expansion limit from 0.10% to 0.08%). Another common modification is to extend the period for monitoring expansion from 14 to 28 days. In some cases both of these modifications are used. For most aggregate sources these changes are unwarranted and increasing the stringency of the limits will not capture, with certainty, all reactive aggregates. (Thomas et al. 2008). The recommended maximum limit of 0.1% expansion at 14 days for the AMBT was established after extensive research that compared AMBT expansion data with expansion in field exposure blocks, field history, and correlation with ASTM C1293 test results (Folliard et al. 2006; PCA 2009; Thomas et al. 2013). Further, the AMBT test is known to be a harsh test and imposing more stringent limits will result in rejection of aggregates or mitigation strategies that may have actually performed well in service. This was shown in the same study previously cited (PCA 2009).

For 142 of the 182 material combinations tested and presented in Table 2, the tests were extended to 28 days rather than the normal 14-day expansion period. The results are summarized in Table 3. It can be seen that in approximately 58% of the cases, both tests agreed. In approximately 40% of the cases the AMBT test produced a false positive while in about 1% of the cases the AMBT test produced a false negative (PCA 2009). Comparing the Table 2 and Table 3 results, it can be seen that a longer test duration resulted in 40% of the aggregates being rejected based on the modified AMBT, but not rejected based on the CPT, which is more than twice the rejection rate seen when the test is conducted without modification.

Another common modification is to use either the AMBT or CPT to test the job mixture of aggregate rather than the individual aggregates. This is a misapplication of the tests and can lead to erroneous results for a variety of reasons. Most fundamentally, all existing ASR tests are empirical tests and by definition, when applied, need to be conducted in same manner as when the supporting data was obtained. Otherwise, the correlation between the various ASR tests cannot be assured. This need for consistency applies to the maximum

expansion limit and to the exposure duration, which are commonly modified as previously discussed. It also applies, however, to the material being tested. The AMBT was developed to test a single aggregate and the CPT was designed to test a combination where one aggregate used is known in advance to not be reactive (i.e., the CPT is testing only one of the aggregates). Neither are applicable to blends of aggregates. Diluting a reactive aggregate with a non-reactive aggregate will provide unpredictable results. To a large degree this is due to the “pessimism effect” associated with ASR as illustrated schematically in Figure 4 (Thomas et al. 2013). In Figure 4, for a given amount of alkali, a threshold amount of reactive silica is required before expansion and cracking occurs. If the silica content is below the threshold amount, ASR could be occurring but it would not be deleterious ASR (Region I in Figure 4). As the amount of reactive silica is increased, and there remains excess alkali, ASR expansion will increase to a maximum (Region II in Figure 4). The maximum in expansion would be considered the pessimum combination of alkali and reactive silica. From this maximum in expansion, as more reactive silica is added, there is no longer sufficient alkali to react all of the silica and expansion is less than maximum (Region III in Figure 4).

Increasing the reactive silica further moves the system to a point where ASR occurs, but quite rapidly, when the concrete is plastic, and no damage occurs (Thomas et al. 2013). Depending on the aggregate and the alkali that is available in the test, adding a non-reactive aggregate dilutant could move the test conditions from Region III towards Region II, or from Region II towards Region I. In the former case there would be an increase in expansion by adding the non-reactive material, in the latter case there would be a reduction in expansion. One study demonstrated this for a number of aggregates where expansion increased as the quantity of non-reactive aggregate increased (Federal Highway Administration 2012).

Testing for ACR Reactive Aggregates

The guides provide information on identifying ACR reactive aggregates. Both guides recommend using ASTM C1293 to assess the reactivity of carbonate aggregates. ASTM C1778 differs from AASHTO R 80 by recommending that carbonate aggregate with expansions greater than 0.04% in one year be examined by a petrographer to determine if the reaction was ACR or ASR, given that ASR could potentially be mitigated. Beyond use of ASTM C1293, both guides recommend using the bulk chemistry of the aggregate as one measure of ACR reactivity, following the approach provided in CSA A23.2-26A *Determination of Potential Alkali-Carbonate Reactivity of Quarried Carbonate Rocks by Chemical Composition*, which is summarized in the guides. The guides also point to use of ASTM C1105, *Standard Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction*.

Table 3. The level of agreement between the one-year CPT and 28-day AMBT conducted in three laboratories using 39 different aggregates and a total of 182 material combinations (PCA 2009)

		ASTM C1260	
		Passed	Failed
ASTM C1293	Passed	23 (16%)	57 (40%)
	Failed	2 (1%)	60 (42%)

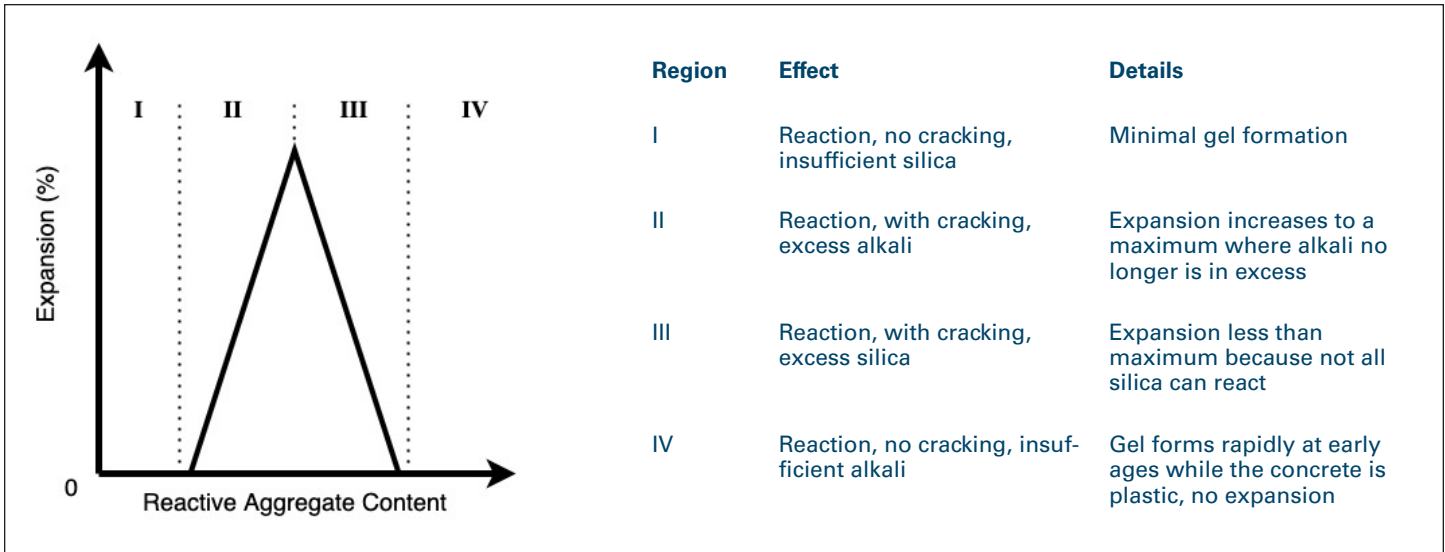


Figure 4. Schematic illustration of the “pessimism effect” associated with ASR (Thomas et al. 2013).

Tests for Evaluating ASR Mitigation Strategies

In the guides, two test methods are used for identifying the effectiveness of an ASR mitigation strategy. The first is ASTM C1293, which is applied in the same manner as when the test is used for testing aggregates, but with two additions. First, the test mixtures are prepared by replacing a portion of the portland cement with the SCM being evaluated. The replacement levels used should cover a range to establish the minimum SCM level appropriate for that particular aggregate (e.g., 20%, 25%, 30%, 35% SCM). It is important to use the same SCM intended for use on the job. When using ASTM C1293, the amount of SCM replacement determined is reliable when the tested aggregate is to be used with a portland cement having an alkali content up to 1.0% Na₂O_{eq}. If the alkali content of the job cement exceeds 1.0% Na₂O_{eq}, the tests should be performed using the job cement and then boosting the alkali content of the test mixture by an additional 0.25% Na₂O_{eq} using NaOH, as described in the test procedure. The second addition is that when using ASTM C1293 to evaluate ASR mitigation strategies, the test duration is two (2) years, not one (1) year as is done when testing only for aggregate reactivity. An expansion of less than 0.04% at two years indicates the materials combination is suitable for use in construction.

The second test is ASTM C1567, *Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method)*. Test method ASTM C1567 is identical to the ASTM C1260 (AASHTO T 303) except the test mixtures are prepared by replacing a portion of the portland cement with the SCM being evaluated. Like with ASTM C1293, the replacement levels used should cover a range to establish the minimum SCM level appropriate for that particular aggregate and the testing is performed using the same SCM intended for use on the job. As for the portland

cement, AASHTO R 80 recommends using the job cement only when the alkali content exceeds 1.0 % Na₂O_{eq}. However, ASTM C1778 does not provide this recommendation and ASTM C1567 states the alkali content of the cement has negligible or only minor effects on the test result.

When using ASTM C1567, the often overlooked step is that previously discussed, ensuring there is correlation between ASTM C1260 (AASHTO T 303) and ASTM C1293 for the aggregate being tested. ASTM C1567 is simply the AMBT with a SCM substituted for a portion of the portland cement. Correlation of ASTM C1567 results with ASTM C1293 results, for the same materials combination, presupposes the underlying AMBT correlates with ASTM C1293 for the aggregate source being used and therefore adequately predicts the aggregate reactivity. If it does not, the resulting assessment of mitigation strategies established by ASTM C1567 testing may also be in error.

Application of the Guide Documents

Overview

The general approaches to AAR outlined in AASHTO R 80 and ASTM C1778 are essentially the same. Both guides derive their technical content largely from the various research projects funded by the FHWA as part of the ASR D&D Program previously mentioned. To apply the guides it is important to understand that it is impossible to completely eliminate the occurrence of AAR in portland cement concrete. The guides were developed to provide approaches that will reduce the risk of deleterious AAR to an acceptable level. As will be discussed later, deciding what constitutes an acceptable level of risk is one of the challenges.

In essence, applying the guides is a two-step process that includes Step 1 - Identify and Characterize the Aggregate

Source, and Step 2 - Select Appropriate Preventive Measures. There are three (3) approaches to accomplishing Step 1 and two (2) approaches to accomplishing Step 2. Each will be summarized in the remaining sections. In the case of identifying and characterizing the aggregate source, it is not uncommon to use all three approaches to ensure the aggregate source is well understood as each approach provides different information that can help the engineer understand the possible risks for ASR associated with using that aggregate source.

Step 1: Identify and Characterize the Aggregate Source

Approach 1 - Establish Field History

The starting point is to select an aggregate source and establish any evidence of field performance. If the performance is satisfactory, the aggregate may be used. Although it seems simple to rely on field performance, it is often difficult for a variety of reasons. First, the concrete used as the example of performance should be at least 15 years old and the aggregates being considered must be the same lithology, to the extent possible, as those to be used in the new concrete. In the case of quarried materials, quarrying operations over a long time period will access different strata of rock that may result in aggregates that differ from the comparison material even though both are from the same quarry, and it is not always clear if that difference is significant. For gravel sources, variability can easily occur throughout a specific deposit. In the same manner, when judging field experience, there needs to be similarities between the composition and content of the cement used in the older and newer concrete. More clearly, the alkali loading of the two concrete mixtures must be the same, or higher in the aged concrete. Other factors such as the amount and type of SCM used in each mixture should also be similar and the concrete being used as the basis of comparison should have similar exposure conditions as the new concrete structure being considered. Both guides recommend examining cores from the comparison structure to confirm the aggregate mineralogy is the same, to identify the type and quantity of SCM used, and to look for any sign of AAR-related damage. If the field performance is deemed satisfactory, the aggregate should only be used in mixtures having an equal or lower alkali loading, an equal or higher SCM content, and equal or less aggressive environment.

Approach 2 - Petrographic Analysis

When establishing an aggregate as being non-reactive, another approach is to conduct a petrographic analysis to determine the aggregate mineralogy and identify the quantity of reactive minerals, if present. ASTM C295 *Standard Guide for Petrographic Examination of Aggregates for Concrete* provides procedures to identify and quantify reactive materials in aggregate. This approach, however, only identifies if reactive

minerals are present. If they are identified, separate testing is required to identify the extent of reactivity and if necessary, establish mitigation measures.

In the case of a carbonate aggregate, petrographic analysis can be very useful to screen potentially reactive aggregates. The ACR-aggregate mineralogical texture is unique and can be identified by petrography. Both guides also recommend the use of bulk chemical analysis of the aggregate to help identify potentially ACR-reactive aggregates. The guides provide a plot of CaO/MgO percent weight ratio versus Al₂O₃ percent weight and identifies regions where potentially reactive aggregates typically plot. As previously stated, if an aggregate is confirmed as ACR reactive, that source must be avoided because ACR cannot be mitigated.

Approach 3 - Aggregate Testing

The approach for aggregate selection most commonly used is to test the aggregate for expansion using the tests previously introduced. The AMBT, is generally preferred because of its short duration. Given the occurrence of false positives, the guides explicitly state that aggregates should not be rejected solely on the basis of AMBT results. Additionally, due to an increasing awareness that false negatives do occur, the guides caution the reader of this inherent risk as well. Also, as previously discussed, both guides warn against using the AMBT to evaluate job mixtures of aggregates.

When relying on the AMBT test, it is necessary to establish the correlation with the CPT test as shown in Figure 3. Therefore, as a SHA puts in place an approach to ASR risk minimization based on these guides, it is not possible to avoid ASTM C1293 completely. In many cases, this requires only limited, but regular, use of the CPT as a QA measure, coupled with routine application of the AMBT either for screening aggregates or assessing mitigation strategies on a job basis.

Step 2: Select Appropriate Preventive Measures

AASHTO R 80 and ASTM C1778 each provide the same two general approaches for developing measures to minimize the risk of ASR; one approach is based on performance in laboratory testing and the other is based on prescriptive requirements. There are, however, notable differences between the performance-based approaches presented in the two guides and if an agency is going to develop a specification that is based on one or the other document, these specific areas should be understood. The performance-based approach is discussed in the following section and the areas of difference are noted.

Approach 1 - Performance-Based Approach

In this approach, mitigation measures are tested using either ASTM C1293 or ASTM C1567; use of ASTM C1293 is encouraged (Thomas et al. 2008). With either test, it is not possible to determine the benefit of adjusting the concrete

alkali loading given the nature of the tests where the concrete is exposed to excess alkali. The SCM to be evaluated is substituted for a portion of the portland cement in a test mixture and it is recommended to prepare and test multiple mixtures with a range of SCM replacement to identify the proper dosage for the job mixture.

The limits for ASTM C1567 are not consistently stated when comparing AASHTO R 80 and ASTM C1778. In the former, an expansion “no greater than 0.10% at 14 days” is used to indicate the material combination is suitable for use in construction. In ASTM C1778 it is stated as requiring a 14-day expansion of “less than 0.10%.” In practice, if the mixture has an expansion of 0.10%, exactly, it would be prudent to increase the SCM dosage and get safely below this expansion limit. However, confusion in statements such as this can lead to confusion in practice. It is expected that future versions of these documents will be harmonized to eliminate these issues².

Also, with respect to the AMBT expansion limits, the supporting research for these guides states that when modified limits or duration have been applied to evaluate the aggregates using the AMBT, those same modified limits should be applied to ASTM C1567 when used to evaluate mitigation strategies (Thomas et al. 2008). Both guides are silent on this point but it should be noted and considered when modified expansion limits are adopted. Notably, ASTM C1778 is clear in stating that more onerous limits “...should not be adopted unless it can be demonstrated that extension of the test period is required to capture aggregates that are believed to be similar to those that have been identified as being deleteriously reactive either by concrete prism testing or field performance.”

In addition to what has already been identified, there are other notable differences between AASHTO R 80 and ASTM C1778 for the performance-based approach. The first difference involves testing SCMs with a high alkali content. AASHTO R 80 is silent on this issue but ASTM C1778 states that ASTM C1567 should not be used when evaluating fly ash with an alkali content greater than 4.0% Na₂O_{eq}, or a slag cement or silica fume with an alkali content greater than 1.0% Na₂O_{eq}. For high-alkali SCMs, only ASTM C1293 is recommended for use. The rationale for this recommendation is presented in the supporting documentation (Thomas et al. 2012). The data set used to determine this limit has been challenged by some for being limited in scope. With new SCMs coming into the market that have relatively higher alkali contents (e.g., high-alkali ground glass), there is renewed interest and ongoing research to establish the efficacy of ASTM C1567 for testing these materials.

The second notable difference between AASHTO R 80 and ASTM C1778 is with regards to using lithium for ASR mitigation. ASTM C1778 provides limited performance-

based guidance for using lithium. The only reference is to CSA A23.2-28A *Standard Practice for Laboratory Testing to Demonstrate the Effectiveness of Supplementary Cementing Materials and Lithium-Based Admixtures to Prevent Alkali-Silica Reaction in Concrete* and to the FHWA D&D research (Thomas et al. 2008). Conversely, AASHTO R 80 integrates the procedure cited in ASTM C1778 as part of its procedure for applying ASTM C1567. Therefore, if lithium nitrate is to be used as a mitigation strategy, it may be advantageous to cite and include the specific guidance provided in AASHTO R 80.

Approach 2 - Prescriptive Approach

Although the performance-based approach using ASTM C1293 provides the best overall guidance, it is widely recognized that in many cases waiting two years for a test result is not possible. It is also recognized that testing based on the AMBT can result in either false positives or worse, false negatives, and these outcomes lead to a risk associated with relying on only rapid testing. For these reasons, the FHWA D&D research also developed a prescriptive approach to ASR mitigation that provides a methodology well-suited for inclusion in a construction specification. Each guide includes this prescriptive approach, which is summarized in this section. There is good agreement between AASHTO R 80 and ASTM C1778 regarding the prescriptive approach, with some minor differences that will be noted.

The prescriptive approach may look confusing at first but it can be summarized as four general steps:

1. Determine the Reactivity of the Aggregate
2. Establish the Tolerable Level of Risk for ASR in the Structure
3. Determine What Level of Prevention is Required
4. Select the Desired Preventative Measure

Determine the Reactivity of the Aggregate – It is necessary to test the aggregate using either ASTM C1260 (AASHTO T 303) or ASTM C1293. The testing is the same as required for the performance-based approach, including the need to verify the agreement between the 14-day AMBT test and the one-year CPT test, if the AMBT test is to be used. Based on the results from this testing the aggregate is placed in one of four aggregate reactivity classes: R0 (Non-reactive), R1 (Moderately reactive), R2 (Highly reactive), R3 (Very highly reactive).

Establish the Tolerable Level of Risk for ASR in the Structure – Based on the aggregate reactivity, exposure conditions and the size of the concrete element, the level of risk for ASR is established. There are six (6) different levels of risk ranging from Level 1 to Level 6. The risk increases for mass concrete, or for concrete exposed to moisture and of course, for concrete exposed to alkalis while in service. Mass con-

crete is defined as any structure with at least one dimension greater than 3 feet (0.9 meters). The threshold for moisture risk is an environment with an average relative humidity greater than 60%. Below this threshold is considered a dry environment. External sources of alkali could result from deicing and anti-icing salts as well as exposure to saltwater, either directly or by airborne mist.

Determine What Level of Prevention is Required – This step generally causes the most confusion as it requires the user to classify the structure being built in terms of the severity of consequences should ASR occur. This results in a structure classification ranging from S1 to S4, which is then combined with the level of risk to establish the level of prevention required for the structure. There are six levels of prevention specified and each is given a letter designation (i.e., V, W, X, Y, Z, ZZ). Note that prevention level V requires no preventative action. Both guides provide descriptions of the consequences as well as the acceptability, and they provide examples for structures in each class.

For example, for S2 the consequences listed are “Some safety, economic, or environmental consequences if major deterioration” and the acceptability description is “Moderate risk of ASR is acceptable.” The examples of S2 structures cited are “Sidewalks, curbs, and gutters. Elements with service life < 40 years.” The suggested design service life is definitive and by itself provides good guidance. However, many engineers would disagree with accepting a “moderate risk” of ASR in sidewalks and would strongly disagree with “Some economic consequences” for ASR in sidewalks and would agree with the S3 description of “Significant economic consequences.” This position is not without merit. For a medium sized city with 100’s of miles of sidewalk, the cost of replacement, should ASR occur, represents a significant economic consequence. Also, there would be clear safety concerns with sidewalks exhibiting widespread map cracking, expansion, spalling or blow ups. The tendency is to err on the side of caution and in the example cited, select S3 rather than S2. Note that S3 is the recommended structure classification for pavements, culverts, low volume bridges, etc.

To continue the example, if the aggregate used is R1 (i.e., AMBT expansion ≥ 0.10 , < 0.30) and the sidewalk is exposed to deicers, the change from S2 to S3 increases the fly ash replacement from 20 to 25 percent, or the slag cement replacement from 35 to 50 percent. In both cases, the impact on curing and finishing need to be considered, particularly for a sidewalk where hand finishing will likely occur. There is an unspoken risk that by trying to reduce the risk of ASR to the absolute minimum, the risk of scaling or other issues might increase given the higher SCM replacement. In the end, the engineer must make the decision but when doing so it is important to keep all aspects of the concrete placement in mind, not just the risk of ASR.

Select the Desired Preventative Measure – The final step is selecting the preventative measure. The guides provide the following three options:

Option 1: Limiting the Alkali Loading of the Concrete

This option for prevention is based on the alkali loading of the concrete which is calculated by multiplying the portland cement content of the concrete by the alkali content of the portland cement. Recall that only the alkali content of the portland cement is included in determining the alkali loading. The relationship between expansion and alkali loading has been demonstrated (Thomas et al. 2012) and is the most effective way to measure the total alkali in a mixture. It also illustrates the value of optimizing a concrete mixture to use as little cement as needed. For example, if the concrete mixture is using 658 lb/yd³, and the cement has a Na₂O_{eq} value of 0.60 percent, then $658 \times 0.0060 = 4.0$. Therefore, the alkali loading of the mixture is 4.0 pounds per cubic yard, the same as obtained with the cement having a Na₂O_{eq} value of 0.77 percent with a cement content of 517 lb/yd³.

When portland cement with an SCM, or when a blended cement is used, the alkali loading is calculated only on the basis of the portland cement fraction. In other words, a 25% replacement of portland cement with fly ash reduces the alkali loading to 75% of the value obtained with portland cement only for the same total cementitious materials content. In the guides, prevention levels W, X, and Y can be achieved by simply limiting the alkali loading.

Option 2: Using SCMs

Because of its simplicity and effectiveness, this is the option for prevention most commonly used. Both guides provide a table that prescribes the minimum SCM level required for prevention levels W, X, Y, and Z. There are some limitations placed on the SCMs if using this prescription table. For fly ash, the CaO content must be 18 percent or less and the Na₂O_{eq} content must be no greater than 4.5 percent (AASHTO R 80) or no greater than 4.0 percent (ASTM C1778). Fly ash not meeting these requirements can be used, but it must be tested using the performance-based approach. The justification for these limits is based on the work of Shehata and Thomas (2000, 2002) and has been summarized (Thomas et al. 2012) as part of the FHWA D&D reports. The research showed that fly ash with a CaO content greater than 20 percent was not effective at replacement levels of 25 percent. Likewise, the same research showed that fly ash with a Na₂O_{eq} content greater than 5% was also not effective at a 25% replacement level. As previously mentioned, the guidance regarding high-alkali fly ash has been questioned by some, and ongoing research is developing a more comprehensive dataset to guide the use of high alkali fly ash. Other SCMs prescribed in the guides include slag cement and silica fume, both having a Na₂O_{eq} content no greater than 1

percent. Silica fume is proportioned based upon the alkali loading of the concrete mixture. The SCM levels prescribed are valid for mixtures with a $\text{Na}_2\text{O}_{\text{eq}}$ content between 0.70 and 1.00 percent. For mixtures with $\text{Na}_2\text{O}_{\text{eq}}$ contents outside this range, the guides provide adjustments to be applied to the prescriptive SCM replacement recommendations.

Option 3: Limiting the Alkali Loading of the Concrete and Using SCMs

For the highest level of prevention (i.e., Level Z and Level ZZ), the guides provide this option which requires combining Options 1 and 2 to use an SCM and limit the alkali loading of the mixture.

Implementation Experience

AASHTO R 80 and ASTM C1778 are guides; they are not specifications. That being said, the information is presented in a manner that readily allows for adoption into a materials specification. To date, only one agency has broadly implemented the guidance (PennDOT, 2020). A number of states have specifications that include portions of the guidance. The Pennsylvania experience has been positively received by industry and it has led to significant improvements in their concrete structures (PACA, 2020). Under their previous specifications, which relied on ASTM C1260 (AASHTO T 303) testing, over 80% of all Pennsylvania aggregates were classified as reactive.

As part of implementing AASHTO R 80, 374 aggregate sources used in concrete in the state were tested using ASTM C1293 (one-year test) and 64% of all tested sources were classified as non-reactive (PACA 2020), a significant change from only 20% being considered non-reactive. In reviewing the PennDOT specification, it is clear that some modification to the guide language was implemented and some other small changes were made based on research with their local materials. This is likely the best way for a SHA to become familiar with the guidance and work towards implementation; through focused research, work through the guide documents and methodically evaluate the state's available materials. By doing so, baseline information can be gathered (i.e., aggregate reactivity data) that can help facilitate implementation.

Closing

The approaches outlined in AASHTO R 80 and ASTM C1778 are currently the best available guidance on AAR. The information presented in the guides has resulted from a multi-year, in-depth study of AAR and the guidance has been demonstrated as effective through block-studies and other field performance. With respect to minimizing the risk of AAR, existing approaches, such as simply limiting the alkali content of the cement, are not as reliable. The guides are not static; research is constantly underway to improve these

documents. As one example of the continuation of research on AAR risk mitigation, the National Cooperative Research Program (NCHRP) is conducting Project 10-103 *Improving Guidance of AASHTO R 80/ASTM C 1778 for Alkali-Silica Reactivity (ASR) Potential and Mitigation*. As part of this research, new tests are being investigated and if proven effective, they will be integrated into the existing guides. Adopting the AASHTO R 80 and ASTM C1778 approach offers the opportunity to significantly improve concrete durability and by adopting a uniform guidance approach, implementation throughout the industry is easier to accomplish and will only serve to improve delivery of durable concrete construction.

Notes

- (1) See <https://www.fhwa.dot.gov/pavement/concrete/asr/resources/index.cfm> for a comprehensive list of publications stemming from the FHWA D&D Program.
- (2) Note that AASHTO and ASTM consistently treat limits differently throughout both guide documents. Whereas AASHTO will set a limit to be inclusive of the limit value, ASTM sets the limits to not exceed the limit value. Example: AASHTO writes ≤ 0.10 percent; ASTM writes < 0.10 percent. This applies to the performance-based limits as well as the prescriptive limits.

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