ALKALI-SILICA REACTIVITY IN THE STATE OF MONTANA

FHWA/MT-21-006/9577-607

Final Report

prepared for
THE STATE OF MONTANA
DEPARTMENT OF TRANSPORTATION

in cooperation with
THE U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

and

THE MONTANA CONTRACTORS ASSOCIATION

September 2021

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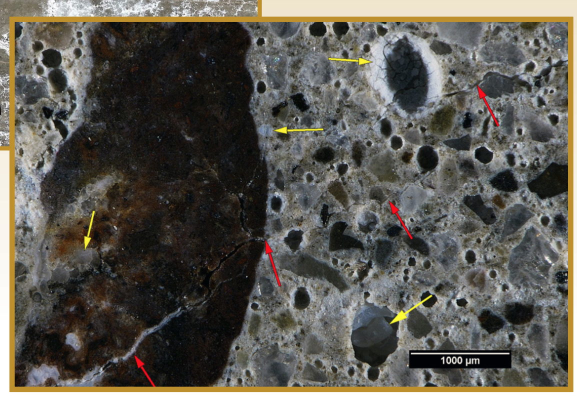

RESEARCH PROGRAMS

Alkali-Silica Reactivity in the State of Montana

Final Report

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16. Abstract

The primary objective of this research was to evaluate the potential for deleterious Alkali Silica Reactivity (ASR) in the state of Montana. In connection with this goal, (1) a literature review was conducted to summarize the ASR practices used by various state departments of transportation, as well as several federal agencies, (2) potential cases of ASR damage in the state were identified and investigated, and (3) existing testing methods were used to test the reactivity of several aggregate sources throughout Montana.

It was found that the neighboring Canadian provinces and all investigated states (sans North Dakota), directly address ASR in their material specifications, to varying degrees. Overall, while there is not an overwhelming amount of evidence of ASR being a major problem in Montana, this research clearly demonstrated the potential for deleterious ASR in the state. Several sites around the state showed distress from ASR, and all of the tested aggregates showed some reactivity.

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UNIT CONVERSIONS

Measurement	Metric	English
Length	1 cm	0.394 in
	1 m	3.281 ft
	1 km	0.621 mile
Area	1 cm^2	0.155 in^2
	1 m^2	1.196 yd^2
Volume	1 m^3	1.308 yd^3
	1 ml	0.034 oz
Force	1 N	0.225 lbf
	1 kN	0.225 kip
Stress	1 MPa	145 psi
	1 GPa	145 ksi
Unit Weight	1 kg/m^3	1.685 lbs/yd^3
Velocity	1 kph	0.621 mph

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1 INTRODUCTION

1.1 Background

Alkali-Silica Reactivity (ASR) is a deleterious reaction that takes place in concrete between alkalis present in the binder and reactive forms of silica in the aggregates. While the alkali ions are primarily attributed to the portland cement, other sources including supplementary cementitious materials, particular types of aggregates, chemical admixtures, and certain deicing solutions can exacerbate the alkali hydroxide concentrations in the concrete mixture [1]. ASR can cause significant damage leading to reduced life span, costly repairs, and/or replacement of the concrete. This damage is initiated by the swelling (in the presence of water) of a gel that forms on the surface of the reactive aggregates, and typically results in map cracking similar to that shown in Figure 1. While ASR has been documented as an issue in many states, little work has been conducted to determine the potential/presence of ASR in Montana.

Figure 1: Typical ASR Crack Pattern

ASR was first documented in 1940 in California [2]. Material testing methods were soon developed to assist in detecting reactive aggregates and to mitigate the potential of ASR in concrete. The initial ASTM specification developed to identify reactive aggregates failed to return consistent results and led to the development of ASTM C1260 and ASTM C1293. Although the current methods are well accepted and used extensively in the concrete industry, these test methods have several shortcomings. ASTM C1260 offers rapid test results, but is often cited as being overly conservative, and has occasionally returned false-negative or false-positive results [3]. ASTM C1293 has proved to be more accurate and less conservative than ASTM C1260; however, its use in practice is hindered by the long duration of the test (at least one year). Due to these issues, recent research has focused on the development and evaluation of shorter duration, more accurate methods for testing for reactive aggregates. In conjunction with the development of testing standards, mitigation methods have also been developed and established to reduce the impact of ASR. These methods include the incorporation of supplementary cementitious materials (SCMs) in the concrete mixture (e.g., class F fly ash, silica fume, and natural pozzolans), and admixtures such as lithium.

While research on ASR in Montana is scarce, the research that has been conducted thus far indicates that some of Montana's aggregate sources are susceptible to ASR [4, 5]. In particular, Lawler and Krauss [5] tested four aggregate sources from geographically diverse areas in Montana, and found three of these aggregates to be reactive, with the fourth being somewhat reactive. They also demonstrated that the reactivity of these aggregates could be mitigated with the use of supplementary cementitious materials. The Montana Contractor Association (MCA) recently sponsored a project to establish a database of ASTM C1260 and C1293 ASR tests that have been conducted in the state by various sources (e.g., aggregate suppliers, contractors, batch plants, and testing companies). However, the researchers in this investigation found that the majority of tests that have been conducted in the state have successfully included supplemental cementitious materials to mitigate potential ASR reactions. Thus, these tests were not useful in determining the underlying reactivity of the aggregates.

Despite the apparent potential for ASR in Montana, no cases of actual ASR have been documented in the state. The lack of documented cases may be due to several factors. First, cases may exist, but have not had an outlet for documentation. Further, ASR occurs in the presence of moisture and the relatively dry Montana climate may be preventing/reducing deleterious ASR. Finally, the use of supplementary cementitious materials and chemical admixtures, which have become more common in conventional concrete mixtures in the state of Montana, may be reducing the presence of ASR.

1.2 Research Objectives

The primary objectives of this research were to evaluate the potential for deleterious ASR in the state of Montana, to develop a testing protocol for identifying potentially reactive aggregates, to identify existing cases of ASR damage in the state, and to test the reactivity of several aggregate sources using existing and newly developed methodologies.

1.3 Scope

The research objectives were achieved through the completion of the following tasks.

- An extensive literature review was conducted to determine ongoing regional and federal ASR practices (Chapter 2). Current and newly developed aggregate testing methods, as well as techniques for identifying ASR in existing structures were investigated and summarized.
- Research was conducted on several existing concrete structures exhibiting ASR-related distress. Several concrete cores were obtained from the sites and further examined in a laboratory setting to determine if ASR was the cause of deterioration and assess the extent of damage. Several of the cores were tested using the Los Alamos Staining Method, and all were evaluated with petrographic analysis. The results of these tests are discussed in detail in Chapter 4 of this document.
- Aggregates from various locations around the state were evaluated in accordance with ASTM C1260 and AASHTO T380. The results of these tests are discussed in detail in Chapter 5 of this document.

2 SUMMARY OF REGIONAL AND FEDERAL PRACTICES

Reactive aggregates and the associated damage resulting from their use are known to be an issue in several Canadian provinces and many states nationwide. Figure 2 [6] shows the states where ASR damage has been documented nationwide. Additionally, results from ASR tests have indicated that many aggregates nationwide have a moderate to high ASR potential [4, 7-9]. The following sections discuss the ASR practices of neighboring Canadian provinces, and several state departments of transportation across the intermountain-west and northern states. Additionally, the practices of several federal agencies are discussed.

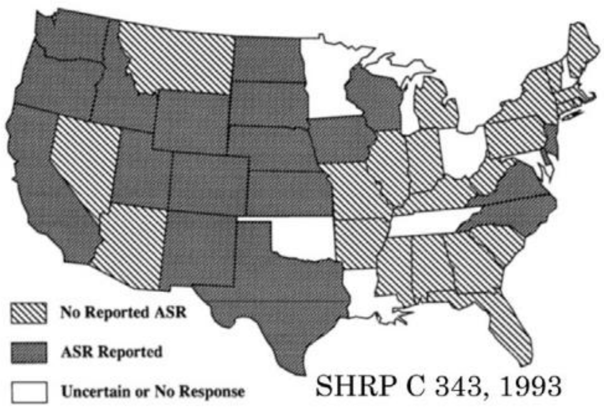


Figure 2: Occurrences of ASR in the United States [6]

2.1 Transport Canada

The Alberta Ministry of Transportation has provisions in their material specifications [10] that address alkali-silica reactivity. The specification indicates that all portland cement must meet the requirements of CSA Standard A3000 (Cementitious Materials Compendium). Unless otherwise specified, Type GU (general use) portland cement is to be used. The corresponding American standard is ASTM C150, Type I. Aggregates are to be evaluated for the potential of deleterious alkali-reactivity in accordance with the CSA A23.2-27A test method. This standard practice is used to identify the degree of aggregate alkali-reactivity as well as mitigation measures.

The British Columbia Ministry of Transportation and Infrastructure standard specifications [11] have provisions that address alkali-aggregate reactivity. The document limits the alkali content of a concrete mix, contributed by the portland cement, to 2.2 kg/m^3 for aggregates classified as moderately reactive and 1.8 kg/m^3 for aggregates classified as highly reactive. Aggregates considered extremely reactive are not permitted for use. Portland cement is required to be Type GU, unless specified otherwise, and shall conform Ato CSA Standard A3000. The alkali content of the cement expressed as (Na_2O + 0.658K_2O), often referred to as Na_2O_{eq}, shall not exceed 0.60% by mass. ASR-specific testing is required for both fine and coarse aggregates. These methodologies include CSA A23.2-14A (Potential Expansivity of Aggregates), and CSA A23.2-25A (Test Method for Detection of Alkali-Silica Reactive Aggregates by Accelerated Expansion of Mortar Bars). The validity of these test results expire every three years.

2.2 Colorado Department of Transportation

The Colorado Department of Transportation (CDOT) has requirements in their standard material specifications [12] that explicitly address ASR-related aggregate testing and mitigation measures. Concrete aggregates are required to be tested for ASR susceptibility using either ASTM C1260 or ASTM C1293. If the aggregate is known to be reactive, ASTM C1567 results can be used in place of the ASTM C1260 results. Should any of these tests indicate reactive aggregate behavior, the aggregates are not permitted to be used unless mitigative measures are employed. The success of the mitigation methods is then evaluated using ASTM C1567. Relatively current (no more than two years old) aggregate testing data is required for the approval of a concrete mix design. Regarding cementitious materials, the specifications require all portland cement to conform to ASTM C150, however there is no indicated limit on alkali content.

2.3 Idaho Department of Transportation

The material specifications of the Idaho Transportation Department (ITD) [13] require that concrete aggregates be tested for potential alkali silica reactivity using either AASHTO T303 (ASTM 1260), AASHTO TP110 (AASHTO T380), ASTM C1293, or ASTM C295. If deemed reactive, a mitigation technique must be approved by ITD and implemented. The efficacy of this mitigation technique must then be verified using ASTM C1567, AASHTO TP 110, or CRD C662.

In addition to addressing ASR in their material specifications, ITD has also recently sponsored several research projects focused on identifying potential reactive aggregates in Idaho [7, 14]. Gillerman and Weppner [7] investigated the relationship between rock type and ASR with the intention of identifying geologic units that are more susceptible to ASR. As part of this research, they sampled 40 different aggregates (representative of the various rock types found in the state) and tested them for reactivity in accordance with ASTM C1260 and ASTM C1293. They found a clear correlation between ASR potential and mapped geology/specific rock types in Idaho, and developed maps documenting this correlation. Mishra and Kassem [14] are currently investigating the efficacy of using the newly developed and accelerated testing protocol AASHTO TP110 for evaluating the ASR potential of reactive aggregates. This testing protocol will be discussed in greater detail in a following section.

2.4 Iowa Department of Transportation

The Iowa Department of Transportation material specifications [15] has provisions that require all portland cement to conform to the requirements of ASTM C150 and limit the alkali content, expressed as Na_2O_{eq}, to 0.60% by mass. There is no language in the specifications that restrict aggregate use or require specific ASR-related testing.

Regarding ASR research, the Iowa Department of Transportation sponsored a research program in 2015 [16] that assessed the effectiveness of using fly ash to mitigate ASR in new and existing concrete due to the alkalis (sodium and potassium) present in fly ash.

2.5 Minnesota Department of Transportation

The Minnesota Department of Transportation (MnDOT) standard specifications for construction [17] extensively addresses ASR. According to the provision, portland cement should be Type I or Type I/II and have an alkali content, expressed as Na_2O_{eq}, less than 0.60% by mass. Additionally, the alkali content of portland cement concrete shall not exceed 3.0 lb/yd^3. The MnDOT has developed a modified protocol for evaluating the reactivity of fine aggregates. This methodology follows the procedures of ASTM C1260 and ASTM C1567, however the aggregate is not crushed according to the gradation requirements of ASTM, rather it is tested in the state in which it arrives. The use of mitigation measures for fine and intermediate aggregates is dependent upon the respective expansion data. Coarse aggregates are tested according to ASTM C1293 to deduce the level of ASR reactivity. The MnDOT specification requires the use of mitigation techniques if the coarse aggregate is identified as quartzite or gneiss. In addition to the specification requirements, the MnDOT Concrete Unit has developed a website (https://www.dot.state.mn.us/materials/concrete.html) [18], dedicated to providing aggregate testing results for various source locations.

2.6 Nebraska Department of Transportation

The Nebraska Department of Transportation (NDOT) standard specifications for highway construction [19] address ASR. Portland cement is permitted to be Type I, Type II or Type III and must conform to the requirements of ASTM C150. Additionally, the equivalent alkali content of the cement, expressed as Na_2O_{eq}, is not permitted to exceed 0.60% by mass. The specification indicates no ASR-related provisions for fine aggregate. Regarding coarse aggregate, ledge rock is to be tested according to ASTM C1260. If the expansion results at 28-days exceed 0.10% the aggregate is to be tested according to ASTM C1567. In the event the mortar bars yield an expansion greater than 0.10%, the coarse aggregate is not permitted for use.

The NDOT has placed a particular emphasis on ASR research in recent years by sponsoring several projects that serve to evaluate Nebraska's aggregate reactivity, aggregate testing procedures, and mitigation techniques for concrete susceptible to alkali-silica reactivity [20-23].

2.7 Nevada Department of Transportation

The requirements of The Nevada Department of Transportation (NDOT) material specifications [24] state that cement shall conform to ASTM C150, with the alkali content, expressed as Na_2O_{eq}, limited to 0.60% by mass. The NDOT prohibits the use of aggregates from sources with a known history of alkali-silica reactivity. For mix design approval, concrete aggregates are tested according to AASHTO T303 (ASTM C1260), with resulting expansions limited to 0.10%.

The NDOT addresses ASR-related testing for bridge rehabilitation projects. The ASR practices identified in the NDOT Structures Manual [25] include ASTM C856, AASHTO T299, and the Los Alamos staining method. These tests are ultimately used to determine the condition of the afflicted concrete and to identify the ASR products in existing concrete.

ASR has been a topic of focus for the Research Division of the Nevada Department of Transportation in recent years, with a publication [26] emerging about mitigating Alkali-Silica Reactivity using local natural pozzolans.

2.8 North Dakota Department of Transportation

The standard material specifications for the North Dakota Department of Transportation (NDDOT) [27] do not specifically contain requirements regarding ASR testing. However, Monte Babok, the head of the material testing division at NDDOT, was interviewed regarding NDDOT ASR procedures. During this interview Mr. Babock stated that, although ASR is not specifically addressed in the specifications, NDDOT does address it in some capacity. Specifically, when a concrete mix design using an unknown/questionable aggregate source is submitted to NDDOT for a project, ASR testing will be performed on the mix design to ensure ASR will not be an issue. While ASR is not known to be a major issue in North Dakota, there have been several documented cases of ASR damage in the past several decades [28, 29].

2.9 Oregon Department of Transportation

The Oregon Department of Transportation material specification [30] requires all portland cement to conform to the requirements of AASHTO M85 or ASTM C150 for low alkali cement. Portland cement of Type I, II, or III is permitted for use. The specification states that the alkali content in the cement, expressed as Na_2O_{eq}, shall not exceed 0.60% by mass. There are no provisions regarding fine or coarse aggregates. The ODOT bridge inspection manual [31] provides tools for identification of concrete exhibiting signs of alkali-silica reactivity and suggests control measures that can mitigate this type of response.

Minimal research has been conducted on ASR; however, in 2014 the ODOT sponsored a program [32] that examined the effects of different deicers on concrete bridge decks and identified ways to mitigate such effects. The research explored how potassium acetate based deicers could contribute to ASR in concrete.

2.10 South Dakota Department of Transportation

The South Dakota Department of Transportation (SDDOT) has provisions in their material specifications that address ASR [33]. Specifically, they require all portland cement to have an alkali content, expressed as Na_2O_{eq}, less than 0.60% by mass and conform to AASHTO M85. Regarding the aggregates, when specified in the plans, the fine aggregates are required to be tested in accordance to ASTM C1260 if the source has not been tested previously, or if the source has changed. Depending on the amount of expansion observed for the source, different mitigation techniques must be implemented. For expansions less than 0.25%, a Type II cement is required. For aggregates with expansions greater than 0.25% but less than 0.40%, a Type V cement is required, and aggregates with expansions greater than 0.40% are not permitted. It should be noted that the provisions do not permit other mitigation strategies (e.g., the inclusion of fly ash or admixtures) to reduce expansions. There are no provisions regarding coarse aggregates.

In regard to research on ASR, the SDDOT sponsored a research program in 2007 [34] that demonstrated the effectiveness of using lithium to mitigate ASR in new concrete and to mitigate the effects of ASR on existing pavements showing signs of ASR damage.

2.11 Utah Department of Transportation

The Utah Department of Transportation material specifications [35] address alkali-silica reactivity. In order to reduce the potential for ASR, UDOT requires the use of Type II portland cement, or equivalent, conforming to AASHTO M85. Blended hydraulic cement substituted for portland cement must be tested according to ASTM C1567 to verify the expansion does not exceed 0.10%. Regarding aggregate composition, fine aggregates are not to exceed the percentage of deleterious substances as specified in AASHTO M6 (ASTM C33). Similarly, the coarse aggregates are limited to the percentage of deleterious substances indicated in AASHTO M80 (ASTM C33). UDOT requires test results for the potential reactivity of both the fine and coarse aggregates to be submitted for approval of a proposed mix design. Aggregate reactivity is tested in accordance with the specifications of AASHTO M6 and AASHTO M80. If the aggregates indicate reactive behavior, the mitigation technique must be tested in accordance with approved AASHTO and ASTM specifications.

Regarding ASR research, UDOT sponsored a project in 2013 [36] that explored the physical and chemical effects of deicers on concrete pavement. The research indicated that ASR can be introduced and exacerbated by the alkalis commonly present in deicers.

2.12 Washington State Department of Transportation

The Washington State Department of Transportation (WSDOT) has fairly prescriptive methods for identifying and mitigating reactive aggregates in concrete, which are outlined in their material specifications [37]. In order to counteract ASR, independent of the aggregates used in the mix, WSDOT limits the alkali content in portland cement to 0.75% by weight, expressed as Na_2O_{eq}. Further, if low alkali cement is required (based on the results of aggregate tests) the alkali content is limited to 0.6% by weight, expressed as Na_2O_{eq}. Limiting the alkali content present in cement reduces the potential for ASR, while not requiring any special testing or batching with supplementary cementitious materials. However, the importance of determining aggregate reactivity in the concrete matrix is crucial in producing a non-deleterious mix.

In regard to aggregates, the specifications require that aggregates be tested per AASHTO T303/ASTM C1260 by the WSDOT materials lab. For aggregates with large expansions (over 0.45%), the mitigation method must be tested in accordance with ASTM C1567 to confirm the efficacy of the prescribed mitigation technique. Once tested, the aggregate source and test results are stored in the WSDOT Materials Lab's Aggregate Source Approval Database (https://www.wsdot.wa.gov/Business/MaterialsLab/ASA.htm) [9], where users can search available aggregate sources based on location and various selection criteria. In addition to the required ASTM C1260 tests, source owners can also opt to test their aggregates via the less conservative one-year ASTM C1293 test protocol.

2.13 Wyoming Department of Transportation

The Wyoming Department of Transportation (WYDOT) material specifications [38] contain provisions that address ASR. According to the specifications, when required by the contract, AASHTO T303 (ASTM C1260) or ASTM 1567 (when fly ash is used) must be performed on a concrete mixture within 12 months of its use. The 14-day expansions on the concrete mixture must be less than 0.10%. If not, ASR must be mitigated through the use of class F fly ash, lithium compounds, or both.

In regard to research on ASR, WYDOT has sponsored several projects focused on the topic. Specifically, one recent project focused on testing aggregate sources from around Cheyenne, Wyoming, where aggregates have been determined to be reactive and problematic in the past [8]. This study used ASTM C1293, ASTM C1260, the Kinetic Method, a modified Chinese Accelerated Mortar Bar Test, and real-time field exposure to evaluate the aggregates. They found that all sampled aggregates were determined to be reactive with the accelerated mortar bar tests. However, these tests are known for being overly conservative, and several of the aggregate sources were found to be only moderately reactive or non-reactive using ASTM C1293. Additionally, WYDOT has recently sponsored a project focused on evaluating treatment options for ASR-affected concrete [39], and is currently sponsoring a project focused on developing an accelerated test method using an autoclave to evaluate ASR potential in concrete that is more reliable than ASTM C1260 [40].

2.14 Federal Highway Administration

The Federal Highway Administration (FHWA) does not require any specific testing for ASR, but rather leaves it to the individual state departments of transportation to establish their own testing/mitigation protocols. However, the FHWA either directly or indirectly sponsors most of the research conducted on ASR. Further, the FHWA maintains a website with a large compilation of ASR research references, and helpful resources (https://www.fhwa.dot.gov/pavement/concrete/asr.cfm) [41].

2.15 Federal Aviation Administration

The Federal Aviation Administration (FAA) directly addresses ASR in their material specifications for the construction of airports [42]. These specifications require that (within 6 months of the project) both fine and coarse concrete aggregates be tested in accordance with both ASTM C1260 and ASTM C1567, with an extended duration of 28 days. If lithium nitrate is to be used, the aggregates shall be tested in accordance with Corps of Engineers (COE) Concrete Research Division (CRD) C662 in lieu of ASTM C1567. In order to be deemed acceptable for use, the recorded expansions for a particular mix at 28 days must be less than 0.10%. If not, the mix must be modified and tested again.

In addition to these fairly stringent specifications, the FAA has also published a handbook to assist in identifying, preventing, and mitigating ASR in existing portland cement concrete pavements [43]. Specifically, this handbook provides a step-by-step procedure on how to identify ASR damage based on field inspection and laboratory investigation and provides guidance in determining reaction severity and appropriate mitigation techniques. This handbook also discusses the use of an ASR condition survey, in which the FAA collects data on documented ASR cases; this data includes aggregate source, mix design, life, average rainfall, etc.

2.16 Summary

This literature review summarized ASR practices used by various state departments of transportation, as well as several federal agencies. It was found that neighboring Canadian provinces, as well as most investigated states (Colorado, Idaho, Iowa, Minnesota, Nebraska, Nevada, Oregon, South Dakota, Utah, Washington, and Wyoming), directly address ASR in their material specifications, to varying degrees. The only state, of those investigated, that did not address ASR in their specifications was North Dakota. Regarding federal practices, the FHWA leaves it to the individual states to determine ASR practices, while FAA was found to have fairly stringent specifications.

3 TESTING METHODS

3.1 Conventional Aggregate Testing Methods

This section discusses the testing methods currently used in practice for evaluating the reactivity of concrete aggregates.

3.1.1 Accelerated Mortar Bar Method – ASTM C1260/C1567 – AASHTO T-303

The accelerated mortar bar test (AMBT) was first developed in 1986 by Oberholster and Davies [44]. After some review and minor procedural changes, the AMBT method was adopted by ASTM in 1994 (ASTM C1260) and is currently the most widely used method for detecting reactive aggregates. ASTM C1567 is very similar to ASTM C1260, but is focused on determining the reactivity of mixtures containing combinations of cementitious materials and aggregates, rather than on just the cement and aggregates. Both tests involve immersing mortar bars in an alkaline solution at 80°C (176°F) for 14 days and monitoring their expansion. While these procedures provide rapid and repeatable results, they expose the aggregates to a fairly harsh environment (with an unlimited supply of alkalis and elevated temperatures) and have been known to be overly conservative when compared to results from other testing procedures (e.g., ASTM C1293) and when compared to field performance. Further, this test method does not allow for testing aggregates at their standard size; that is, the aggregates must be crushed and sieved to produce a sand with suitable size and gradation for the mortar bars.

3.1.2 Concrete Prism Test – ASTM C1293

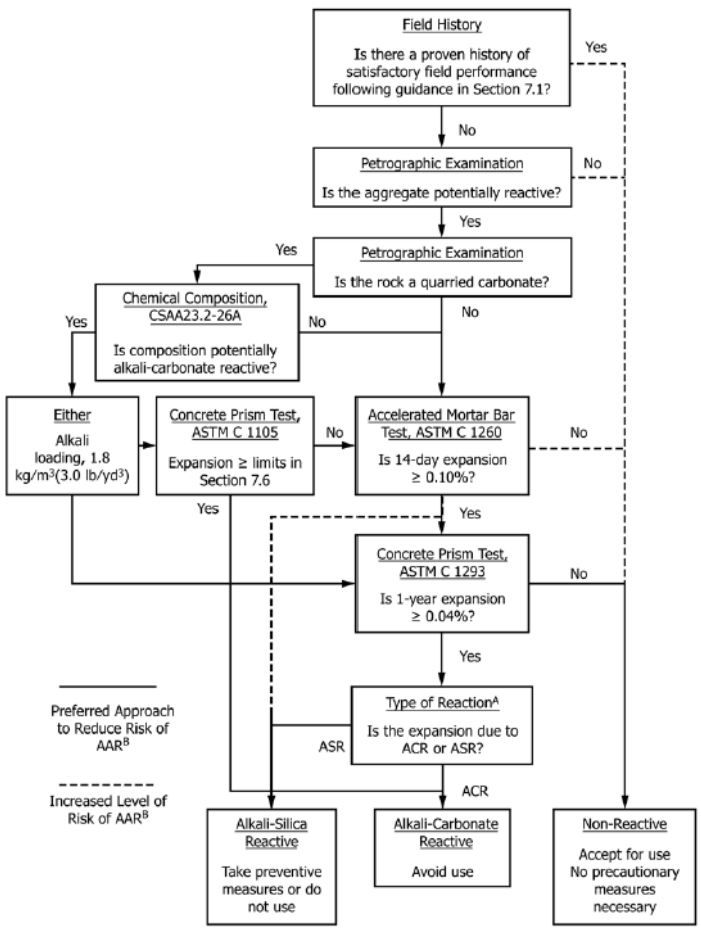
The concrete prism test (CPT) was adopted by ASTM in 1995 (ASTM C1293), following its initial development as a Canadian test method [3]. This test method is known to be one of the more reliable test methods available for testing the reactivity of aggregates; however, its extended timeframe (1-2 years) limits its use in industry. This test procedure consists of casting concrete prisms with an increased alkali content, and then exposing these prisms to high humidity and elevated temperatures for 1-2 years. The expansion of these prisms is then monitored systematically over the duration of the test. This method has been shown to have better agreement with field performance than ASTM C1260, most likely due to its less harsh/more realistic exposure conditions, and the limited supply of alkalis. That being said, it has been known to produce false positive results [3]. Additionally, it should be noted that this test methodology allows for testing coarse aggregates in their standard size due to the larger 75-mm by 75-mm cross-section.

3.1.3 Petrographic Evaluation – ASTM C295

Concrete aggregates can be examined for potentially reactive constituents using petrographic techniques. This petrographic examination should identify and quantify alkali-silica reactive constituents within the aggregates, such as opal, chalcedony, cristobalite, tridymite, highly strained quartz, microcrystalline quartz, cryptocrystalline quartz, volcanic glass, and synthetic siliceous glass. Based on the findings, additional tests may be recommended to confirm these findings.

3.1.4 ASTM C1778/AASHTO R80

Both ASTM and AASHTO recently released similar standards (ASTM C1778 [45] and AASHTO R80 [46], respectively) for determining the reactivity of concrete aggregates (in regard to both ASR and alkali-carbonate reactivity, ACR) and selecting appropriate mitigations measures. These standards do not introduce new testing procedures, but rather provide guidance on how to test for reactive aggregates using existing methodologies (i.e., petrographic examination, accelerated mortar bar tests and concrete prism tests). They also provide guidance on how to interpret test results and take appropriate measures, which include accepting an aggregate for use, avoiding using an aggregate, and/or mitigating reactivity. Mitigation methods for ASR can be selected using either a prescriptive or performance-based methodology, and since the potential for deleterious reactions depend on both the concrete mix design and the in-service exposure of the concrete, guidance is provided based on the type of structure and the exposure environment. The mitigation methods include limiting the alkali loading of the concrete, using supplementary cementitious materials, using lithium admixtures, or a combination of these methods. The flow chart provided in Figure 3 [45] summarizes the procedure introduced in these standards. It should also be noted that both standards state that if there is a proven history of satisfactory field performance then the aggregate source may be accepted for use with no precautionary measures. Regarding this, the standards also provide guidance for conducting a field survey of existing structures that were constructed using the same or similar aggregates, and that were exposed to similar conditions as the structure to be constructed.



[A] The type of reaction only needs to be determined after the concrete prism test if the aggregate being tested is a quarried carbonate that has been identified as being potentially alkali-carbonate reactive by chemical composition in accordance with test method CSA A23.2-26A.
[B] The solid lines show the preferred approach. However, some agencies may want to reduce the amount of testing and accept a higher level of risk and this can be achieved by following the direction of the hashed lines.

Figure 3: ASTM C1778/AASHTO R80 Flow Chart [45]

3.2 Newly Developed Aggregate Testing Methods

Several efforts are underway to develop aggregate testing methods that are more reliable than ASTM C1260, but are shorter in duration than ASTM C1293. The following subsections briefly discuss these methodologies.

3.2.1 Miniature Concrete Prism Test – AASHTO T-380

The most promising aggregate testing methodology under development is the Miniature Concrete Prism Test (MCPT). This methodology is based off of research conducted by Latifee and Rangaraju [1] in 2015, and was recently adopted by AASHTO in 2018 [47]. This testing methodology is a hybrid between ASTM C1260 and ASTM C1293, using a similar testing procedure to ASTM C1260 with the mix design specified by ASTM C1293. The test specimens for the MCPT (cross-section dimension of 50 mm) are larger than those used in ASTM C1260 (25 mm) and smaller than those used in ASTM C1293 (75 mm). This size allows for testing coarse aggregates without the need for crushing, sieving, and combining. Regarding duration, the MCPT can characterize aggregate reactivity in 56 days, which is longer than the 14 days used in ASTM C1260, but significantly shorter than the 1 year required for ASTM C1293. It should also be noted that this test methodology can be used to test the potential for supplementary cementitious materials and admixtures to mitigate ASR expansions.

This methodology is fairly new and takes significantly less time to conduct, but it has been shown to have good correlation with ASTM C1293 (the most reliable method) test results and field performance. In the original research conducted by Latifee and Rangaraju [1], aggregates with known field performance were tested with the MCPT methodology, and results were then compared to the results obtained from ASTM C1260 and ASTM C1293. The 56-day expansions recorded from the MCPT correlated well with the 365-day expansions recorded from ASTM C1293, with an R^2 value of 99% (Figure 4). It should also be noted that the Idaho Transportation Department is currently funding a research project focused on evaluating the advantages associated with implementing the MCPT within their specifications to quantify the ASR potential of aggregate sources in Idaho [14].

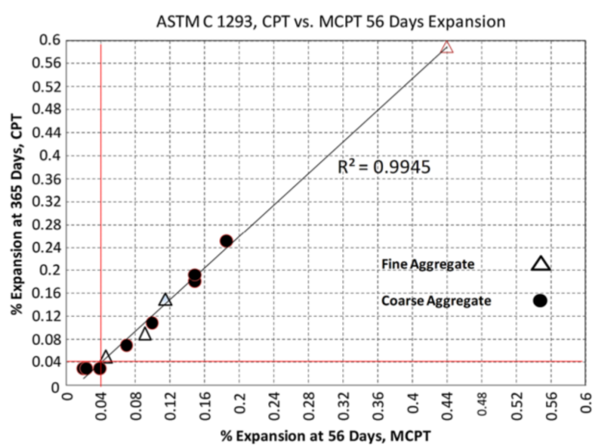


Figure 4: Correlation Between ASTM C1293 and MCPT Expansions [1]

3.2.2 Chinese Accelerated Mortar Bar Test

The Chinese Accelerated Mortar Bar Test (CAMBT) method is a combination of the Chinese Autoclave Method [48] and the standard accelerated mortar bar method. While this procedure has been shown to have good correlation with the accelerated mortar bar method, correlations with the CPT have been shown to be very poor [49], indicating that further research is necessary to establish the full reliability of the test. Similar to the shortcomings of the accelerated mortar bar method, the poor correlation may stem from the unrealistic and harsh testing environment to which the specimens are exposed.

3.2.3 Rapid Chemical Method for Determining Alkali-Silica Reactivity

Mukhopadhyay [50] proposed using a volumetric change measuring device to detect the amount of volume change due to ASR. The proposed method takes 5 days to complete instead of the standard 14-day AMBT testing procedure. The procedure involves placing aggregates in test chambers, heating them to specific temperatures in the presence of different alkalinity solutions, and measuring the changes in the pressures. This will yield activation energy and a threshold alkalinity level, which will allow for the determination of aggregate reactivity level as well as a recommendation for mitigation procedures outlined in the report. Tied to the Rapid Chemical Method, a kinetic-based classification system has been investigated using the data from the test [51]. This method demonstrates potential to be used as an alternative to ASTM C1260. However, more testing is required to establish its efficacy.

3.2.4 Autoclave Tests for Determining Potential Alkali-Silica Reactivity of Concrete Aggregates

The U.S. Army Engineer Research and Development Center (ERDC) recently developed a test methodology that is very similar to the ASTM C1260, where an autoclave is used to accelerate reactions. This test methodology is called the Five-Hour Autoclave Test [52]. In this methodology, mortar bars of the same dimensions and mix proportions as ASTM C1260 specimens are cured for 48 hours before autoclaving at 130 ℃ for 5 hours, at which time expansions are observed. The researchers found that this test methodology obtained similar results to ASTM C1260 in 85% of 20 samples tested and agreed with results from ASTM C1293 in 100% of 10 samples tested. However, further research is required to establish this test methodology as a viable alternative to standard ASR testing methods.

Similarly, a test method has been developed that uses an autoclave to accelerate reactions in ASTM C1293 concrete prisms [53]. During its development this testing methodology was shown to correlate well with ASTM C1293 and ASTM C1260 test results and was shown to be superior to ASTM C1260 with regard to speed and accuracy. An ongoing research project at the University of Wyoming is focused on further developing and evaluating this testing methodology [40].

3.3 Methods for Identifying and Quantifying ASR Damage in Existing Concrete

The following sections discuss the methods available for identifying and quantifying ASR damage in existing concrete.

3.3.1 Los Alamos Staining Method

The Los Alamos Staining method [54] can be used to determine the presence of ASR in hardened concrete. This test involves placing various dye chemicals on the surface of fractured concrete, and depending on the nature of the reaction products present, the ASR gels will change color. This method provides a simple means for evaluating the extent and distribution of the gel products associated with ASR and is easy to interpret (even for those not experienced in petrographic analysis). However, more accurate/thorough testing methods, such as petrographic analysis, should be performed to fully characterize the damaged concrete.

3.3.2 Petrographic Analysis

The most accurate way of determining the presence and extent of ASR in hardened concrete is petrographic analysis. Some of the common petrographic techniques are listed below.

- *Comprehensive petrographic examination per ASTM C856*; this technique analyzes the condition of concrete represented by an extracted, full-depth core. This test examines air entrainment, determines depth of potential carbonation, characterizes cracks/microcracks, identifies rock types that are commonly susceptible to ASR, and identifies secondary deposits such as ASR gel and ettringite, which is commonly abundant in concrete with ASR.
- *Scanning electron microscopy (SEM) with energy-dispersive x-ray spectrometry (EDS) per ASTM C1723*; along with higher magnifications, this test allows for in-situ chemical analyses of materials in a microstructure, and this test can positively identify if ASR gel is present.
- *Damage Rating Index (DRI) analysis*; this analysis inventories indicators of ASR in the concrete cores by subdividing cores into 25 x 25 mm (1 x 1 in.) cells and tabulating the number of various features that are associated with ASR, including (reaction rims, debonded aggregates, deposits of gel in voids, deposits of gel in microcracks, etc.). This provides a method to compare levels of deterioration between different cores.

3.3.3 Non-Destructive Methods for Determining Damage Caused by ASR in Concrete

The U.S. Department of Energy Office of Nuclear Energy’s Light Water Reactor Sustainability (LWRS) Program, aimed at extending the life of nuclear power plants, conducted research on non-destructive evaluation (NDE) methods for testing for the presence of ASR damage [55]. In their research, they used ultrasound to evaluate concrete slabs with varying levels of ASR. They found that the same technique developed for testing and quantifying freeze-thaw damage in concrete, the Hilbert Transform Indicator (HTI), may also be successful in testing for ASR damage. This research determined that, while these two types of testing both correlate to damage in concrete in some ways, they do not determine the depth or distribution of the cracks/damage.

Another group of researchers from various universities around the country conducted similar research on ultrasonic NDE techniques to evaluate damage in concrete resulting from ASR [56]. This study used three parameters of the ultrasonic testing as an attempt to measure ASR damage in concrete: wave speed, attenuation, and the amplitude of waves. The study found that wave speed and attenuation had poor correlation to ASR damage because of a lack of sensitivity. The amplitude of waves was more accurately used to quantitatively track ASR damage in concrete, and this parameter correlated well with the reduction of compressive strength resulting from ASR.

In another study, acoustic emission (AE) was used to test for ASR damage in concrete specimens [57]. AE tracks the generation of waves produced from a sudden release of energy, such as cracking or expansion of concrete, and a subsequent redistribution of stress. Using highly sensitive sensors, internal cracks in concrete can be detected before they become visible. In this research, the activity from AE correlated well to the rate of expansion observed for concrete specimens expanding due to ASR and correlated well to petrographic analysis.

3.4 Summary

Existing testing methodologies were discussed along with their shortcomings. ASTM C1260 is the most used methodology, but has been shown to have poor correlation to field performance. ASTM C1293 is the most accurate method, but is not used as often due to its prolonged duration (1-2 years). Several new testing methodologies are currently under development to overcome the shortcomings of these existing methodologies. The most promising of which is the miniature concrete prism test, which has recently been adopted by AASHTO as an aggregate testing methodology. This methodology was shown to have good correlation with ASTM C1293, but only takes 56 days to complete.

4 CASES OF ASR DAMAGE IN MONTANA

To assist in identifying potential cases of ASR damage in the state, a document (included in Appendix A) was prepared and sent to all major entities responsible for the construction and maintenance of Montana's concrete infrastructure. This memo resulted in several organizations coming forward with potential concrete damage that may be attributed to ASR. Specifically, Morrison and Maierle, the Department of Natural Resources and Conservation (DNRC), and MDT all reported potential cases of ASR in existing structures around the state. Four cases were identified and investigated, including two at the Billings Logan International Airport, one at the Willow Creek Dam near Harrison, MT, and one case in a bridge over Belt Creek just southeast of Belt, MT. This chapter details each of the reported sites, the laboratory studies conducted on the afflicted concrete, and the results of each ASR analysis.

4.1 Billings Logan International Airport

Two sites at Billings Logan International Airport were identified by Morrison and Maierle Consulting Engineers as having potential ASR damage. To determine if ASR was the cause of the concrete deterioration and to assess the extent of the damage, several concrete cores were obtained from the sites and further examined in a laboratory setting. Specifically, the cores were initially tested using the Los Alamos Staining Method, followed by a comprehensive petrographic analysis.

4.1.1 Site Description

The two sites investigated at the Billings Logan International Airport were located on the apron, where aircraft are parked, loaded, unloaded, refueled, and maintained. Figure 5 shows the locations of the two sites. Little information was available regarding the composition of the concrete (mix design, aggregate source, mitigation measures, etc.) at each site. Additionally, no information was provided regarding the original construction specifications or records of testing completed during or after construction. However, one primary distinction between the two sites was the age of the original concrete construction. Specifically, the concrete age at Site 1 and Site 2 were 40 years and 20 years, respectively. Exposure to weather, deicing agents, and other moisture was probable at both sites. Due to the proximity to the terminal, Site 1 likely experienced significant exposure to deicing salts, which can advance the concrete distress caused by ASR. Site 2 was located near a drain, thus the continual contact with moisture likely progressed the ASR damage. Figure 6 illustrates the extensive cracking damage at each site.

Figure 5: Billings Logan International Airport Core Sites

(a) Site 1 (b) Site 2

Figure 6: Concrete Crack Patterns

4.1.2 Coring Procedure

A Core Bore M-Series Core Drill Rig equipped with a Husqvarna 4 inch wet/dry diamond core bit was used to extract the specimens from each site. The setup can be seen in Figure 7a below. Four specimens were extracted from each site (Figures 8 and 9); of which, three were removed from areas with visible surface patterned cracking and one from an area that showed no signs of distress. The cores were approximately four-inches in diameter and represent a partial thickness of the apron slabs (6 ¼ - 8 in.) in length/depth. Figure 7b shows examples of the extracted cores and an example cored hole is shown in Figure 7c. Following extraction of the cores, the holes were filled with high strength grout as shown in Figure 7d.

(a) Core Drilling

(b) Extracted Cores

(c) Cored Hole

(d) Refilled Core Hole

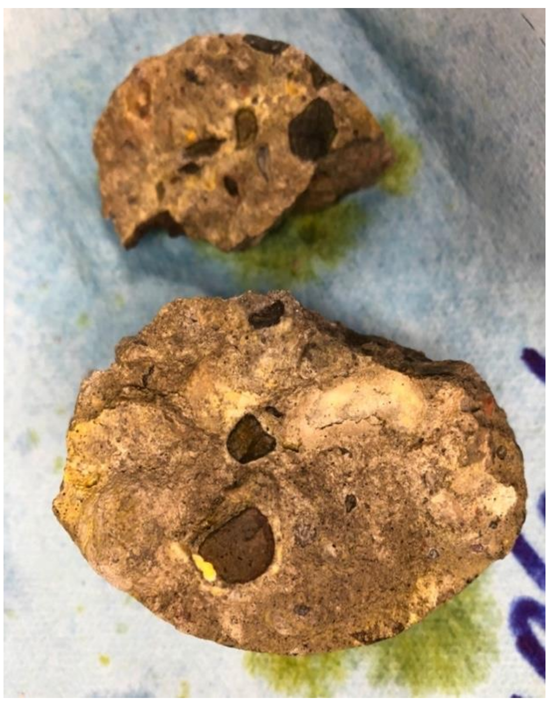
Figure 7: Core Extraction Process

Figure 8: Site 1 Coring Locations

Figure 9: Site 2 Coring Locations

4.1.3 Los Alamos Staining Method

The Los Alamos Staining Method [54] was used to establish the existence of ASR in the concrete cores. Two cores, one from each site, were tested according to this method. To start, distilled water was applied to the freshly broken surface of each test specimen. Next, the yellow reagent was applied to the test surface and subsequently washed off with distilled water spray. The surface was then dried and examined for the presence of ASR gel, indicated by a bright yellow stain. The procedure was completed again on the same test surface using the red reagent. In this case, bright pink with yellow stains were indicative of advanced ASR. The results of the initial round of staining are shown in Figure 10. The bright yellow stain surrounding the aggregate particles and in the voids of the concrete signify that presence of ASR gel in both specimens. The results of the subsequent round of staining, shown in Figure 11 appear to confirm the presence of advanced ASR. It should be noted that the results of this test method were subjective, and in no way definitively confirm the presence of ASR. However, after reviewing the results of the petrographic analysis, and gaining an understanding of the signs of ASR damage, it was evident that these cores were affected by ASR. That said, the extent of the damage cannot be conclusively characterized as severe/advanced based on the results of this method.



(a) Site 1

(b) Site 2

Figure 10: Los Alamos Staining – Yellow

(a) Site 1 (b) Site 2

Figure 11: Los Alamos Staining - Pink

4.1.4 Petrographic Examination

Two cores, one from each site, were sent to DRP, A Twining Company, for petrographic analysis per ASTM C856 [58] to assess the presence and extent of the ASR damage in the concrete represented by the cores. These cores were denoted 1a and 2a, corresponding to Sites 1 and 2 respectively. It should be noted that Core 1a was referred to as Sample 2 and Core 2a was referred to as Sample 5 in the report prepared by DRP [59], available on the project website (https://www.mdt.mt.gov/research/projects/mat/alkali_silica.shtml). The results of the petrographic analysis indicate the presence of severe ASR in both cores.

The characteristics of the cores indicate that similar concrete mixtures were used in the original construction with minor differences in the cement/cementitious materials and aggregate size. The paste in Core 1a was comprised solely of hydrated portland cement, whereas Core 2a consisted of hydrated portland cement and fly ash. Neither sample contained slag cement or other supplementary cementitious materials. The alkali content of the hydrated portland cement was not indicated in either sample. The coarse aggregate used in both concrete mixtures was a crushed siliceous gravel, with reactive components including rhyolite, granite rocks, quartzite, and chert with minimal amounts of andesite and basalt. The coarse aggregate nominal top size was 25 mm (1 in.) and 19 mm (3/4 in.) in Core 1a and Core 2a respectively. The fine aggregate used in both sites was a natural sand with a similar reactive constitution to that observed in the coarse aggregate.

Based on the findings of the report, the damage observed in the concrete cores was due to three cracking mechanisms. These included early age shrinkage cracking, sub-horizontal cracking, and cracking and microcracking due to ASR. The early age shrinkage cracking was likely due to drying shrinkage cracking, which primarily occurs because of capillary water loss. The sub-horizontal cracking was common in both cores and was associated with freeze-thaw damage. Although early age shrinkage and sub-horizontal cracking are distinct failure mechanisms, both expedite the existence of alkali-silica reactivity. The ASR-related cracking and microcracking was significant in both cores. ASR gel deposits were detected within the cracks and surrounding the aggregates. Figure 12 shows a reflected light photomicrograph of the polished surface of Core 1a. In these images, microcracks (red arrows), filled with ASR gel, can be seen cutting through various aggregate particles. The yellow arrows indicate ASR gel either lining a void or within an aggregate particle. Similarly, the red arrows, shown in Figure 13 (Core 2a), indicate microcracks filled with ASR gel, while the yellow arrows reveal a significant ASR gel deposit in the middle of the core. As a result of the findings, the ASR in both cores was rated severe according to a criteria developed by DRP (Table 1) and Stage V using an alternative evaluation scheme (Table 2).

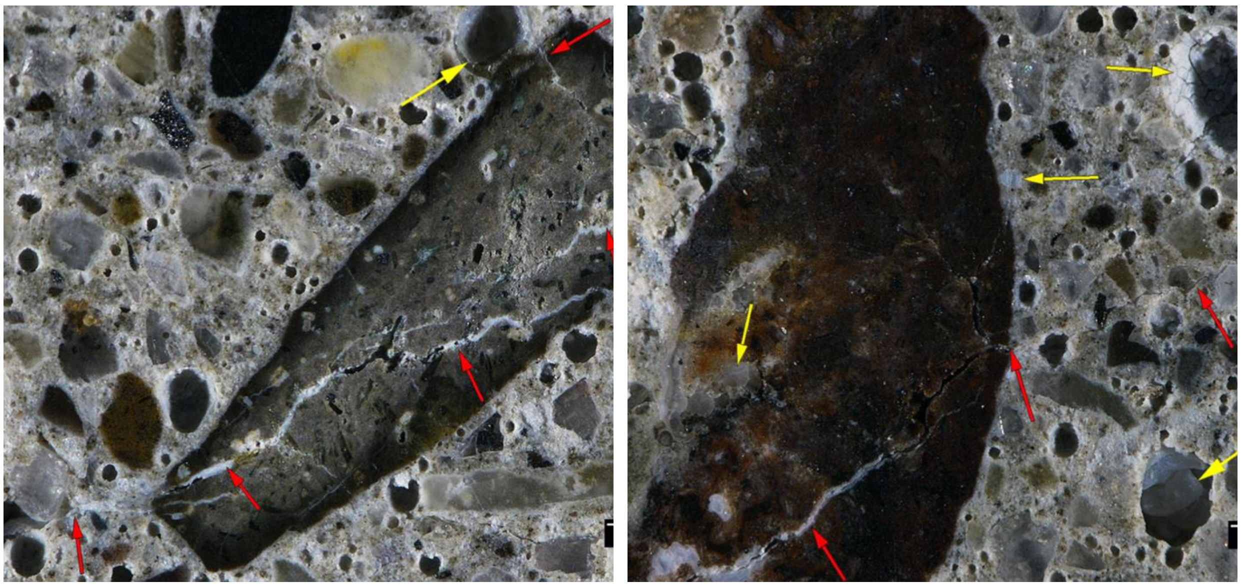


Figure 12: Core 1a Microcracking (Red Arrows) and ASR Gel (Yellow Arrows) [60]

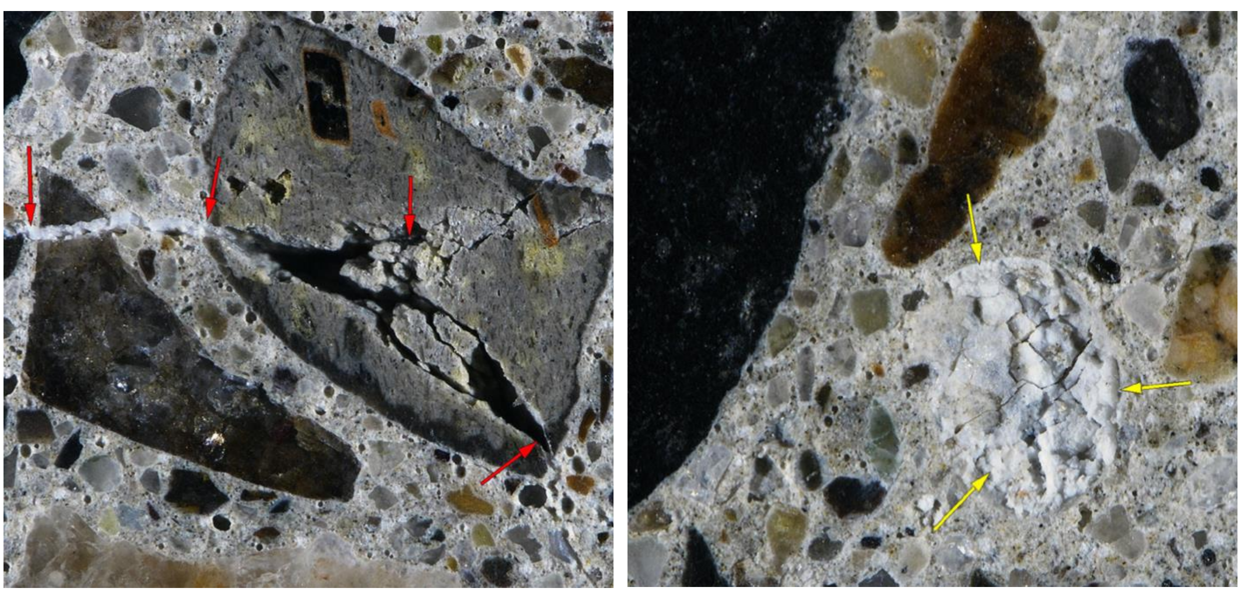


Figure 13: Core 2a Microcracking (Red Arrows) and ASR Gel (Yellow Arrows) [60]

Table 1: DRP Criteria For Severity of ASR Damage [60]

Severity	Criteria
Absent	No reaction rims, microcracks or cracks associated with ASR
Negligible	Only reaction rims observed
Trace	ASR gel rarely observed lining voids near aggregate particles
Minor	ASR gel in voids near reactive particles or rimming reactive particles; microcracks with ASR gel rarely to occasionally observed
Moderate	Microcracks filled with ASR gel commonly observed
Severe	Cracks (> 100 μm wide) due to ASR observed

Table 2: Petrographic Stages of ASR, Katayama et al. [60]

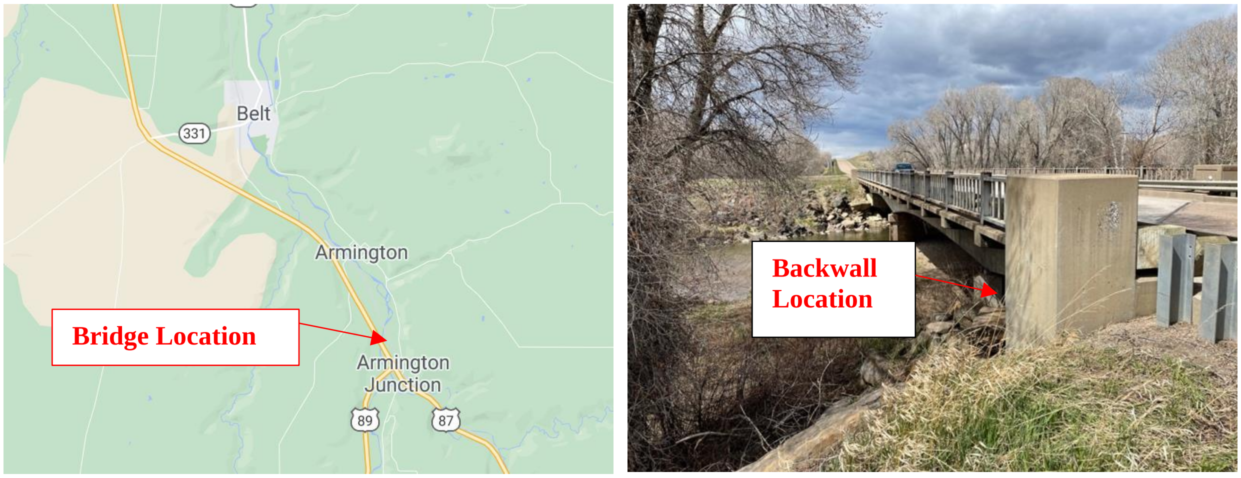
Stage	Criteria
I	Formation of reaction rims within aggregate particles.
II	Exudation of ASR sol/gel around aggregate particles. Darkening of paste around aggregate particles.
III	Cracking of aggregate; ASR gel may line or fill crack.
IV	Propagation of cracks from reacted aggregate into paste. ASR gel may line or fill crack; crack width grows.
V	Filling/lining of ASR gel into distant air voids. Note crack width for advanced damage.

4.1.5 Summary

The findings of both the Los Alamos Staining Method and the petrographic analysis verified the occurrence of ASR in the concrete represented at both Billings Logan International Airport coring sites. The ASR was rated severe and was either directly or indirectly associated with the cracking mechanisms identified in the cores.

4.2 US-87/US-89/MT 200 Bridge over Belt Creek

Potential ASR damage was identified by MDT on a bridge on US-87/US-89/MT 200 over Belt Creek southeast of Belt, MT. The site location and the location of the damage relative to the bridge are shown in Figure 14. One of the backwalls and girders were observed to have significant cracking and efflorescence, as shown in Figure 15. The bridge was built in 1954, is 33.5 ft wide, and has an overall span of 156 ft. Little information was available regarding the composition of the concrete (e.g., mix design, aggregate source, mitigation measures). Additionally, no information was readily available regarding the original construction specifications or records of testing completed during or after construction. Based on the petrographic examination (discussed below), ASR was determined to be a factor in the deterioration. This section describes the coring process, and the results from this investigation.



(a) Bridge Location (b) Backwall Location

Figure 14: Bridge and Backwall Location

(a) Backwall (b) Backwall and Girder

Figure 15: Cracking and Efflorescence on Backwall and Girder

4.2.1 Coring Procedure and Core Locations

A Core Bore M-Series Core Drill Rig equipped with a Husqvarna 4 inch wet/dry diamond core bit was used to extract three cores from the backwall, in the approximate locations shown in Figure 16a. Also, this figure shows one of the researchers preparing an anchor for the core drill, which is shown in the anchored position in Figure 16b. The backwall coring is shown in Figure 17a, while Figure 17b shows one of the extracted cores. Following extraction, the holes were filled with high-strength grout as shown in Figure 18. The cores were approximately four-inches in diameter and varied in depth between 3 and 6 inches.

(a) Coring Preparation

(b) Anchored Core Drill on Backwall

Figure 16: Coring Preparation and Anchored Core Drill

(a) Backwall Coring

(b) Extracted Core

Figure 17: Backwall Coring and Extracted Core

Figure 18: Grouted Core Holes

4.2.2 Petrographic Examination

Two cores from the bridge were sent to DRP, A Twining Company, for petrographic analysis per ASTM C856 [58] to assess the presence and extent of the ASR damage in the concrete represented by the cores. Only one of the cores was analyzed, but two were sent so that the petrographer could choose the best one for analysis. The full report documenting this analysis [61] is included on the project website (https://www.mdt.mt.gov/research/projects/mat/alkali_silica.shtml).

This analysis determined the composition of the concrete to be as follows. The paste contains hydrated portland cement with no fly ash or other supplemental cementitious materials. The concrete is marginally air-entrained with 3-4% total estimated air. The coarse aggregate is a natural gravel with a 50 mm (2 in.) nominal top size although most particles are less than 19 mm (3⁄4 in.) across. The aggregate consists of siliceous and carbonate rocks that include limestone, siliceous limestone, quartzite, chert, and siliceous volcanic rocks. The fine aggregate is a natural sand that consists of rocks similar to those in the coarse aggregate.

The findings from this scope of work indicate that the concrete shows moderate to severe damage from ASR based on a qualitative scale from DRP (Table 1). The progression of the reaction is at Stage IV of six stages using the scale of Katayama (Table 2). The ASR involves siliceous limestone, quartzite, and granitic rocks in the coarse and fine aggregate. Figure 19 shows the light photomicrographs of the polished surface of the main crack in the core. The area shown in this figure is approximately 20 mm (3⁄4 in.) from the outer surface of the bridge. The red arrows highlight deposits of ASR gel that line the walls of the crack. Figure 20 are photographs of the inner fracture surface of the core showing detail of ASR gel deposits around aggregate particles.

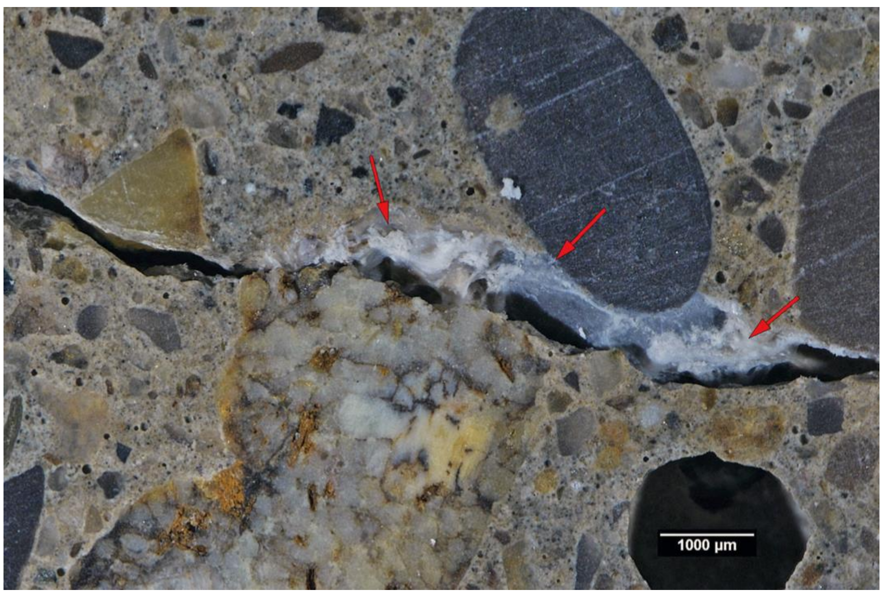
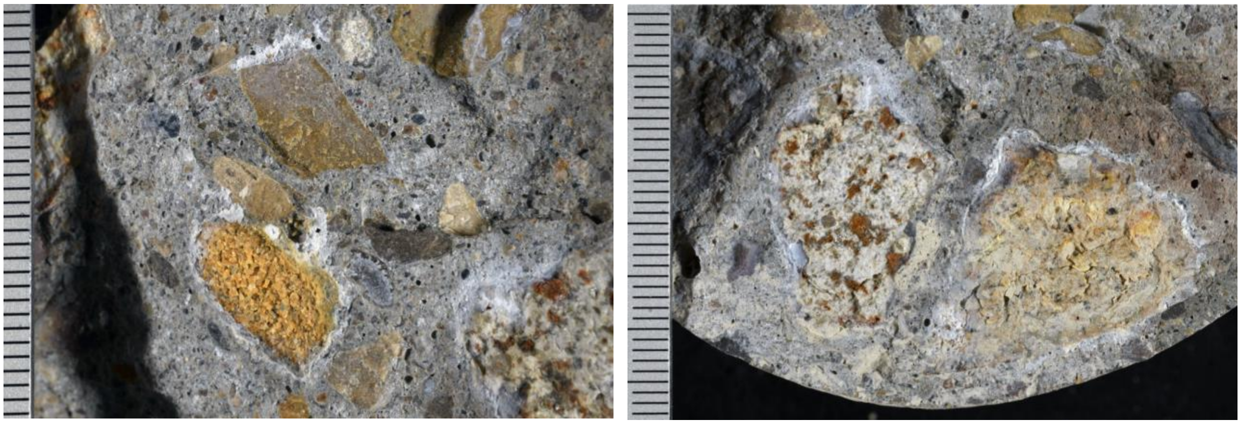


Figure 19: Reflected Light Photomicrographs of the Polished Surface



(a) (b)

Figure 20: Photographs of the Inner Fracture Surface of the Core

4.3 Willow Creek Dam

The Willow Creek Dam Spillway, located four miles east of Harrison, MT, was subjected to investigation after years of deterioration, restricted use, and failed repairs. In 2018, the Montana Department of Natural Resources and Conservation (DNRC) retained Wiss, Janney, Elstner Associates, Inc. (WJE) to assess the condition of the spillway by means of material laboratory examination. The results of the report [62] prepared by WJE are summarized below with the comprehensive report found on the DNRC website: http://dnrc.mt.gov/divisions/water/projects [63].

The 75-year-old, slab-on-grade concrete spillway consists of a 240-ft long chute, side walls, ogee crest, and lower flip bucket (Figure 21). The original construction of the spillway generally remained intact and has only undergone some minor repairs and sealant work over the past 75 years. Information regarding the composition of the original concrete (mix design, aggregate source, mitigation measures, etc.) was not provided. WJE conducted a site visit to visually identify and assess the damage in each component of the spillway. The spillway chute presented with both transverse and longitudinal cracking, spalling, paste erosion, and control joint faulting. Additionally, many of the previously installed repairs had failed. Transverse cracking was observed on each section of the spillway side walls. Unsound concrete, referring to delamination or incipient spalls, and paste erosion were common at this location as well. Both the ogee and the flip bucket, located at the top and base of the spillway respectively, exhibited signs of patterned cracking. In addition to the cracking and other forms of concrete deterioration, several small voids, and one large void (2,900 ft^2.), were detected at various locations beneath the slab using ground penetrating radar (GPR). Given the nature of the structure, exposure to moisture and weather was significant.

(a) Chute

(b) Side Walls

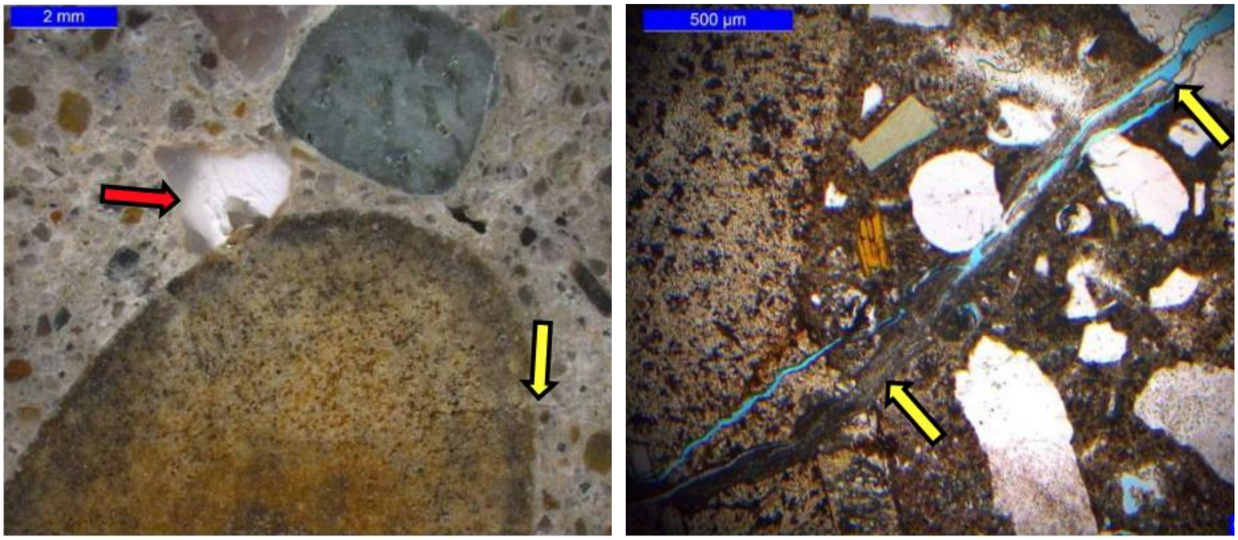
(c) Ogee

(d) Flip Bucket

Figure 21: Spillway Components [62]

Using ASTM C856, both new and previously obtained core samples were examined to characterize the concrete and establish the reasons for deterioration. Eight cores were extracted in 2017 by Pioneer Technical Services, and four additional concrete cores were extracted by WJE during their site investigation in 2018. Seven of the twelve samples were selected for petrographic analysis, including five slab cores and two cores from the spillway ogee. The paste was comprised solely of portland cement. No fly ash, slag cement or other supplementary cementitious materials were identified. The coarse aggregate identified in the concrete was a natural siliceous gravel, with components including felsic igneous and metamorphic rocks, quartzite, chert, and sandstone with minimal amounts of carbonate. The maximum size of coarse aggregate observed was 25 mm (1 in.). The fine aggregate was a siliceous sand.

ASR-related distress is visible in two of the spillway cores shown in Figure 22 below. In Figure 22a, ASR gel deposited in a void is identified by the red arrow, while the yellow arrow points to a crack extending outward from the aggregate particle. Figure 22b shows cracks (yellow arrows), filled with ASR gel, extending between aggregate particles.



(a) Chute Core (b) Ogee Core

Figure 22: Cracking (Yellow Arrows) and ASR Gel Deposits (Red Arrows) [62]

Based on the findings of the report, the damage observed in the concrete cores was due to transverse and longitudinal cracking, patterned cracking, freeze thaw distress, and alkali-silica reaction. Reactive forms of aggregate, cracking and efflorescence were commonly observed in this analysis and can be associated with ASR. The severity of the ASR distress observed in the spillway varied based on location. Advanced ASR was detected in the ogee, while areas of the slab exhibited ASR-related distress to a lesser extent.

5 AGGREGATE TESTING

Concrete aggregates from several locations across Montana were tested according to two testing methods. Specifically, ASTM C1260 (Accelerated Mortar Bar Test) and AASHTO T380 (Miniature Concrete Prism Test) were used. As stated previously, the AMBT is a 14-day test that involves extensive preparation of the aggregate to achieve a specified gradation. While this test yields rapid results, this method has the unfortunate disadvantage of delivering overly conservative results. In contrast, the results of the MCPT are available after 56-days of testing and are comparable to ASTM C1293 with respect to accuracy of test results and field performance. This chapter documents the materials, mix proportions and results of each aggregate test conducted in accordance with both the ASTM C1260 and AASHTO T380 testing procedures.

5.1 Materials

The materials used in this research were prescribed by the testing standards, including aggregate, cement, deionized water, and sodium hydroxide. This section provides details of the mix ingredients used in this study.

5.1.1 Aggregates

Eight aggregates, sourced from various locations around Montana, were tested in this research with the intent to determine the level of aggregate reactivity. These aggregates, of unknown reactivity, were provided by the Montana Department of Transportation. Of which, four were considered coarse aggregates and four were identified as fine aggregates. All selected aggregates were classified as combined/intermediate concrete aggregates. In the ASTM C1260 tests, the fine and coarse aggregates were tested separately. In the AASTO T380 tests, an aggregate of unknown reactivity was tested in conjunction with a non-reactive aggregate of differing type. That is, the reactivity of a fine aggregate was tested with a non-reactive coarse aggregate, while the reactivity of a coarse aggregate was tested with a non-reactive fine aggregate. Graded Ottawa Test Sand was selected as the non-reactive fine aggregate in the MCPT test, and limestone from the GCC Cement Plant in Trident, Montana was used as the non-reactive coarse aggregate. The physical properties for the fine, coarse, and non-reactive aggregates are shown below in Table 3, Table 4, and Table 5, respectively. Included in each table is the aggregate ID, location, oven-dried specific gravity, absorption, and dry-rodded unit weight.

Each aggregate was given a specific identification code that was dependent upon either particle size or reactivity. Fine aggregates were designated with an (F), coarse aggregates with a (C) and non-reactive aggregates with an (NR). Also included in each table are the pit locations of each aggregate. It should be noted that the Knife River Billings – Sindelar Pit was tested twice. This source served as the blind duplicate for this research. The duplicated aggregates were F-2, F-3, C-2, and C-3. The oven-dried specific gravity, denoted by SG_{OD}, was used in calculating the proper mix proportions for both testing methodologies. The aggregate absorption was used to calculate the moisture content correction for the AASHTO T380 mix designs. The dry-rodded unit weight, denoted by DRUW, was used in determining the coarse aggregate, AASHTO T380 mix designs.

Table 3: Fine Aggregate Physical Properties

Aggregate ID	Location	SG_{OD}	Absorption (%)
F-1	Helena Sand & Gravel - Lake Helena Drive Pit	2.59	2.67
F-2	Knife River Billings - Sindelar Pit	2.65	1.38
F-3	Knife River Billings - Sindelar Pit	2.72	1.40
F-4	Knife River Missoula - Allen Pit	2.72	1.61

Table 4: Coarse Aggregate Physical Properties

Aggregate ID	Location	SG_{OD}	DRUW (kg/m^3)	DRUW (lb/ft^3)	Absorption (%)
C-1	Helena Sand & Gravel - Lake Helena Drive Pit	2.52	1609.78	100.50	2.14
C-2	Knife River Billings - Sindelar Pit	2.58	1650.15	103.02	1.27
C-3	Knife River Billings - Sindelar Pit	2.57	1653.07	103.20	1.16
C-4	Knife River Missoula - Allen Pit	2.60	1639.80	102.37	1.15

Table 5: Non-Reactive Aggregate Physical Properties

Aggregate ID	Location	SG_{OD}	DRUW (kg/m^3)	DRUW (lb/ft^3)	Absorption (%)
F-NR-1	Graded Ottawa Test Sand - ASTM C778	2.65			1.76
C-NR-1	CRH Cement Plant - Trident, MT	2.64	1657.15	103.45	2.73

5.1.2 Cementitious Materials

Type I-II portland cement was used in this study, per ASTM and AASHTO specifications. The cement was obtained from the GCC Cement Plant in Trident, MT. The alkali content of the cement used was reported at 0.42% by mass, expressed as Na_2O_{eq}. The autoclave expansion of the cement was below the prescribed limit, at 0.02%. The other chemical and physical properties of the cement used in this experiment conformed to ASTM C150 for Type I-II and Type V, as can be seen in Table 6. The specific gravity of the cement used was documented at 3.15.

Table 6: Chemical and Physical Properties of Portland Cement, ASTM C150

Chemical Properties		
Item	Limit	Result
SiO_2 (%)	NA	20.6
Al_2O_3 (%)	6.0 max	4.0
Fe_2O_3 (%)	6.0 max	2.8
CaO (%)	NA	64.2
MgO (%)	6.0 max	2.5
SO_3 (%)	3.0 max	3.1
Loss on Ignition (%)	3.0 max	2.7
Insoluble Residue (%)	0.75 max	0.4
CO_2 (%)	NA	1.7
Limestone (%)	5.0 max	4.0
$CaCO_3$ in Limestone (%)	70 min	98.0
Inorganic Processing Additions (%)	5.0 max	1.2
Potential Phase Compositions:		
C_3S (%)	NA	58.0
C_2S (%)	NA	15.0
C_3A (%)	8.0 max	6.0
C_4AF (%)	NA	8.0
$C_3S + 4.75C_3A$ (%)	NA	86.5
Physical Properties		
Air Content (%)	12.0 max	29.75
Blaine Fineness (m2/kg)	260 min	413.00
Autoclave Expansion	0.80 max	0.0
Compressive Strength (Mpa) (psi):		
3 days	12.0 (1740)	27.2 (3940)
7 days	19.0 (2760)	35.4 (5130)
Initial Vicat (minutes)	45-375	141
Mortar Bar Expansion (%) (C1038)	NA	
Heat of Hydration (kJ/kg) (cal/g)		
7 days	NA	

5.1.3 Other Required Materials and Reagents

Type IV grade of reagent water was used as the mix water in this experiment, per ASTM D1193. The water was prepared by means of distillation.

Technical grade sodium hydroxide pellets, obtained from Fisher Chemical, were used to generate a 1N NaOH soak solution for immersion of specimens throughout the duration of both the AMBT and MCPT. Additionally, sodium hydroxide was used as an admixture in the AASHTO T380 tests to boost the alkali content of the concrete to 1.25% Na_2O_{eq} by mass of cement.

5.2 ASTM C1260 – Accelerated Mortar Bar Tests

This section documents the procedures used for and results from the ASTM C1260 (Accelerated Mortar Bar Test) tests on the aggregates identified in the previous section.

5.2.1 Test Setup

To begin, the relative density (specific gravity) and absorption of the fine and coarse aggregates were determined per ASTM C128 and ASTM C127, respectively. This information was necessary to calculate the proper mix proportions. Upon completion of the additional ASTM tests, each aggregate was crushed (when needed), sieved, and recombined to achieve the gradation requirements indicated in Table 7.

Table 7: ASTM C1260 Gradation Requirements [64]

Sieve Size		Mass, %
Passing	Retained on	
4.75 mm (No. 4)	2.36 mm (No. 8)	10
2.36 mm (No. 8)	1.18 mm (No. 16)	25
1.18 mm (No. 16)	600 µm (No. 30)	25
600 µm (No. 30)	300 µm (No. 50)	25
300 µm (No. 50)	150 µm (No. 100)	15

In terms of mix proportions, the prescribed cement quantity needed to cast three specimens was 440 grams, and the water-cement ratio was 0.47 by mass. The aggregate quantity for the test mortar was based on the oven-dried relative density of the test aggregate. The results of the ASTM C127 and C128 tests indicated that all eight aggregates had specific gravities exceeding 2.45, rendering each aggregate quantity an even 990 grams. The material proportions used for each AMBT can be seen in Table 8.

Table 8: ASTM C1260 Mix Proportions

Variable	Amount
w/c Ratio	0.47
Reagent Water (g)	206.8
Portland Cement (g)	440
Aggregate - Coarse/Fine (g)	990

A Hobart A200 bench top mixer, seen in Figure 23, was used to mix the mortar in this research. The mortar bar molds used for the AMBT produced test specimens with dimensions of 25 mm x 25 mm x 285 mm (1 in. x 1 in. x 11.25 in.). Metal gauge studs were inserted into each end plate of the mold apparatus as shown in Figure 24. The gauge studs were cast into the specimens to assist with expansion measurements.

Figure 23: Hobart A200 Mixer

Figure 24: ASTM C1260 Mortar Bar Molds

The mortar for each aggregate was mixed in accordance with ASTM C305. Each mix design produced three specimens per aggregate, for a total of 24 specimens. The filled molds were placed in the moist room to cure for a period of 24 hours. After demolding, an initial reading was taken using the length comparator shown in Figure 25 and Figure 26, and the specimens were placed in a water bath at 80°C (176°F) for an additional 24 hours. Following the water bath, a zero reading was taken and the specimens were placed in an approved container (Figure 27) and immersed in the 1N NaOH solution at 80°C (176°F) for a period of 14 days. Subsequent expansion measurements were taken at 1, 3, 7, 10 and 14 days.

Figure 25: ASTM C1260 Specimen

Figure 26: Length Comparator with Reference Bar

Figure 27: Approved Mortar Bar Container

5.2.2 Standard Expansion Limits

ASTM C1260 provides the following expansion limits to assist with interpretation of the test results, shown below in Table 9. Specimens that experience an average expansion less than 0.10% at the end of the 14-day testing period are deemed innocuous or non-reactive, while those exceeding 0.20% are considered reactive. If the expansion results lie within the 0.10 and 0.20% range, the aggregate is potentially deleterious and may require additional evaluation to determine its true behavior.

Table 9: ASTM C1260 Expansion Limits [64]

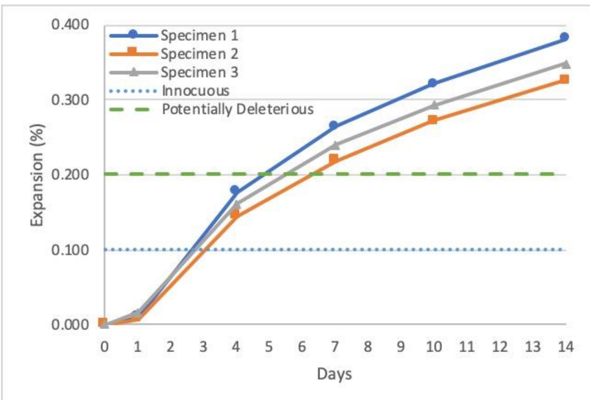
Degree of Reactivity	Expansion (%)
Innocuous	< 0.10
Potentially Deleterious	0.10-0.20
Reactive	> 0.20

The Miniature Concrete Prism Test, requires the use of non-reactive fine and coarse aggregates. AASHTO T380 prescribes graded Ottowa Test Sand for use as the non-reactive fine aggregate; however, no information was provided about a suitable non-reactive coarse aggregate. As a result, an additional ASTM C1260 test was conducted on limestone, provided by the CRH Cement Plant in Trident, Montana, to determine its level of reactivity and its viability for use as the non-reactive coarse aggregate in the AASHTO T380 tests. In addition to the eight fine and coarse aggregates tested, limestone was tested according to the ASTM C127 and C1260 procedures described above.

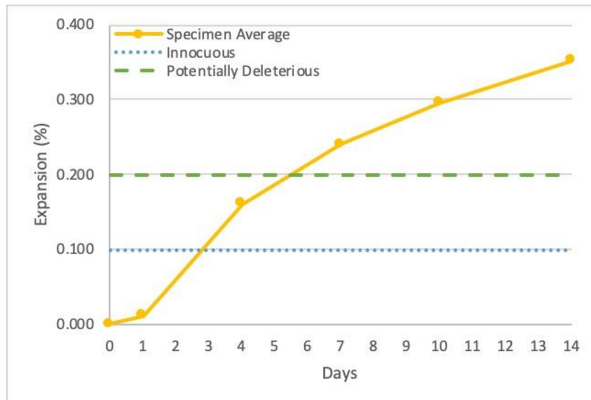
5.2.3 Results from ASTM C1260

The percent expansion versus time for the ASTM C1260 tests is plotted in Figure 28-Figure 32. The plots on the left show the expansion results for all three specimens across a specific mix, while the plots on the right show the average specimen expansion for that specific mix design. The dotted blue line shows the expansion limit for innocuous behavior. Expansion values exceeding the green dashed line at the end of the 14-day testing period are considered reactive. Any value between the dotted/dashed lines is considered potentially deleterious.

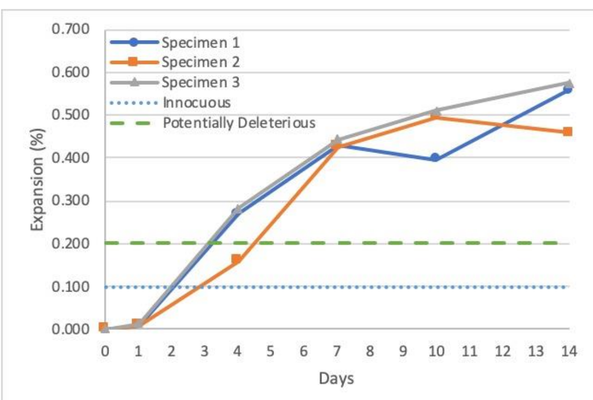
Figure 28 and Figure 29 provide the expansion results for all the fine aggregates tested according to ASTM C1260. As can be seen from these graphs, all four of the fine aggregates exceeded the 0.20% expansion limit and depicted reactive behavior.



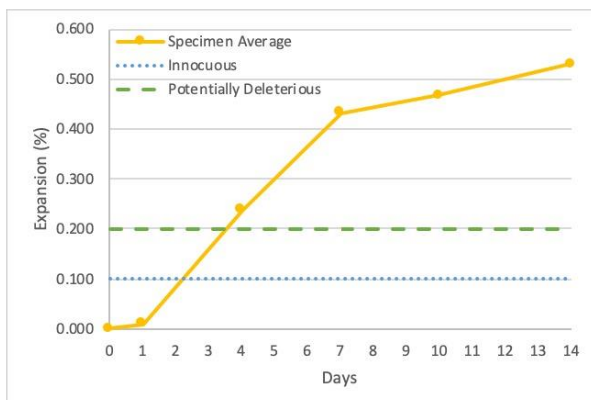
(a) Mix F-1: All Specimens



(b) Mix F-1: Specimen Average

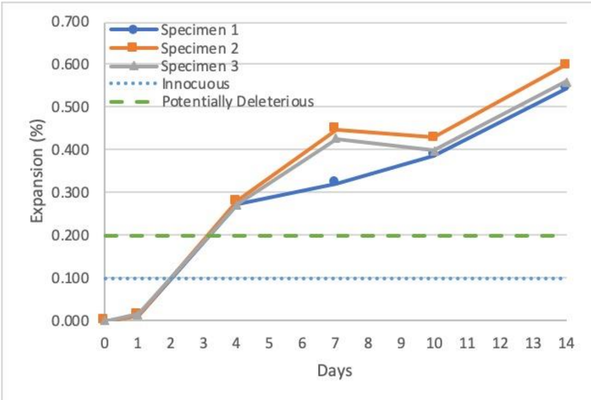


(c) Mix F-2: All Specimens

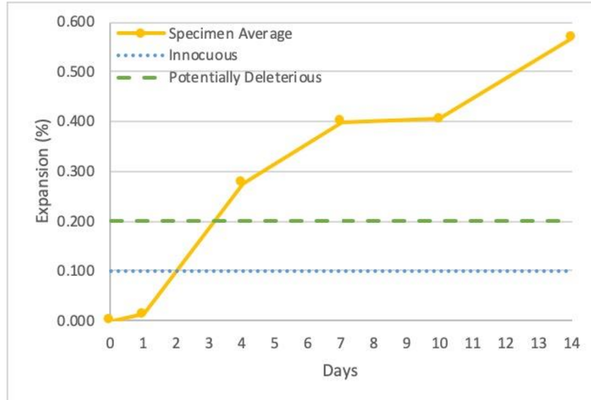


(d) Mix F-2: Specimen Average

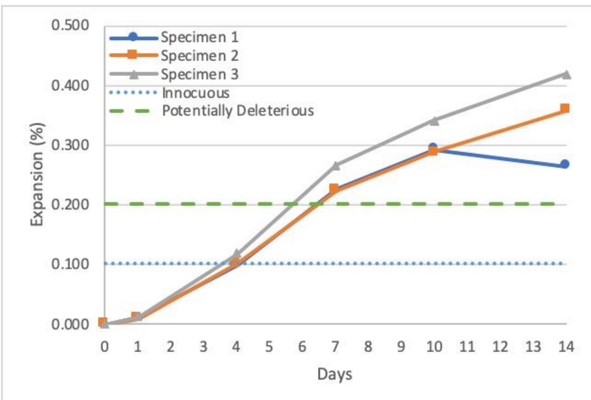
Figure 28: ASTM C1260 Results for Mixes F-1 and F-2



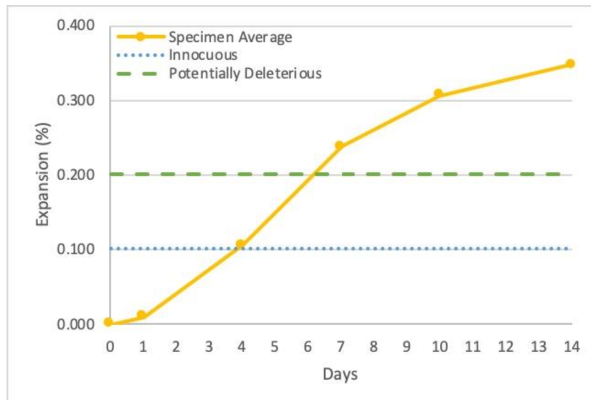
(a) Mix F-3: All Specimens



(b) Mix F-3: Specimen Average



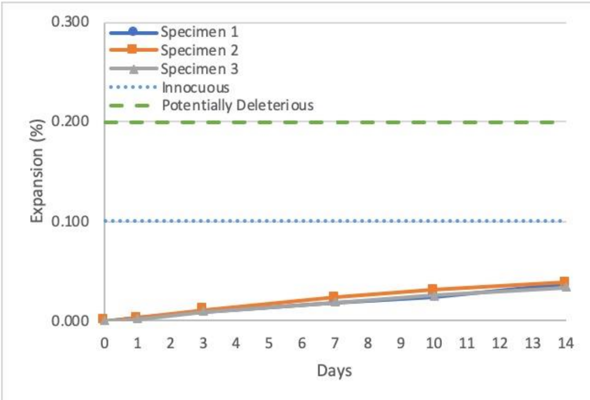
(c) Mix F-4: All Specimens

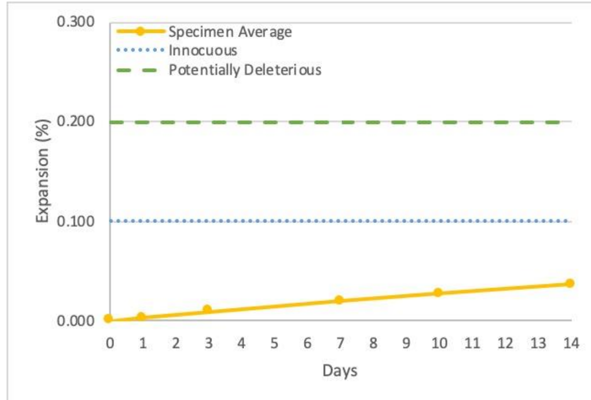


(d) Mix F-4: Specimen Average

Figure 29: ASTM C1260 Results for Mixes F-3 and F-4

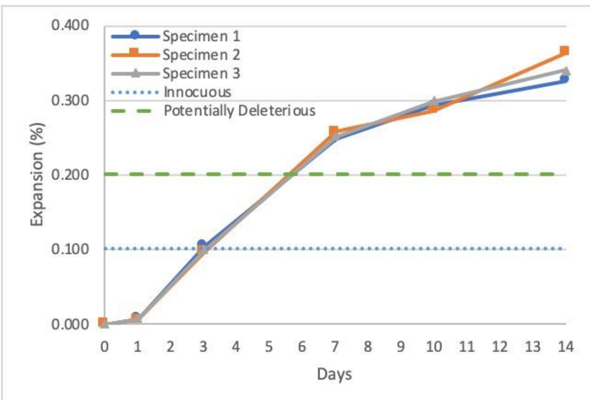
Figure 30 and Figure 31 provide the expansion results for all of the coarse aggregates tested according to ASTM C1260. As can be seen from these graphs, coarse aggregates C-1 and C-4 remained innocuous throughout the 14-day testing period. However, the data showed an upward trend, particularly in the C-4 plot, which could potentially indicate long-term ASR susceptibility. Conversely, coarse aggregates C-2 and C-3 exceeded the 0.20% expansion limit by the 7-day measurements and showed reactive behavior.

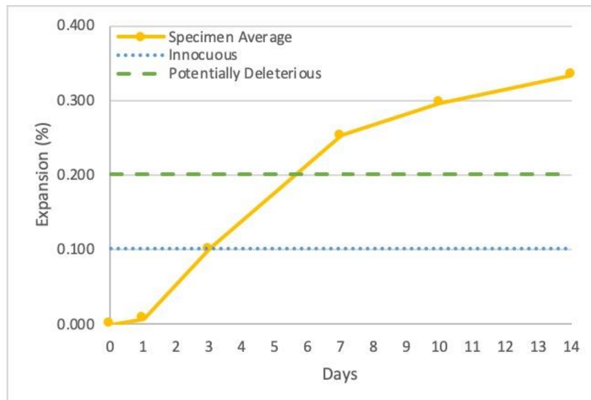




(a) Mix C-1: All Specimens

(b) Mix C-1: Specimen Average

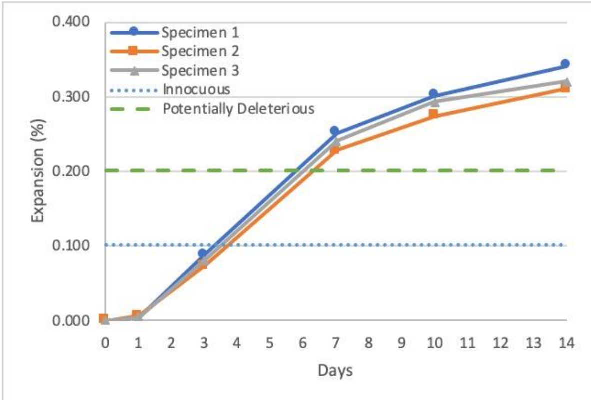




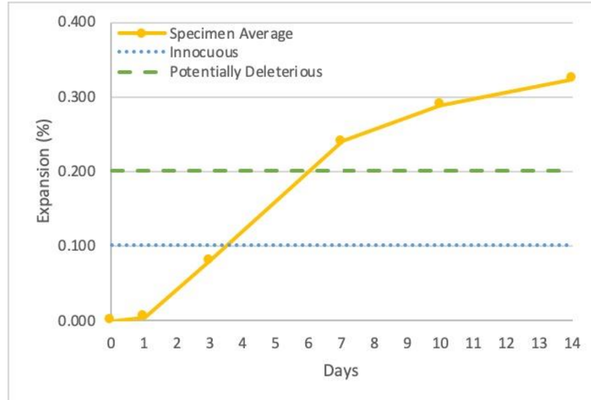
(c) Mix C-2: All Specimens

(d) Mix C-2: Specimen Average

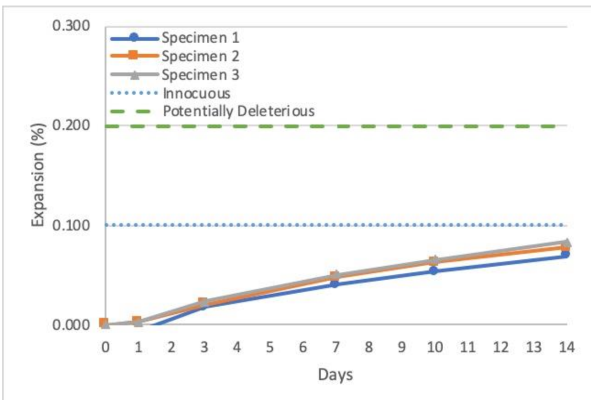
Figure 30: ASTM C1260 Results for Mixes C-1 and C-2



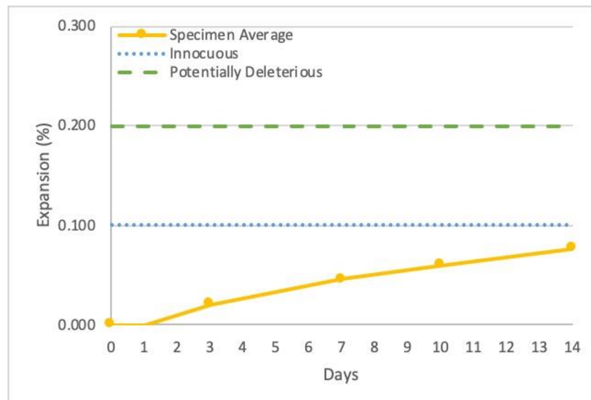
(a) Mix C-3: All Specimens



(b) Mix C-3: Specimen Average



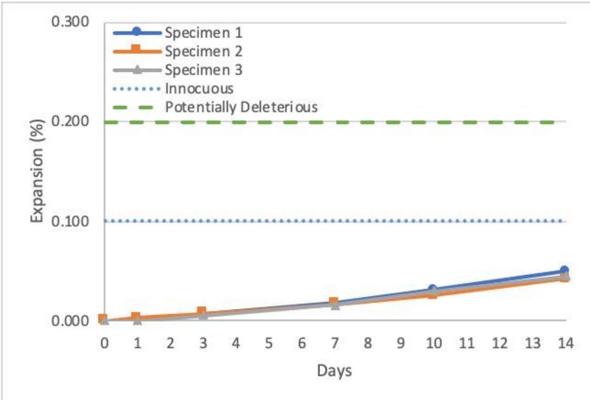
(c) Mix C-4: All Specimens



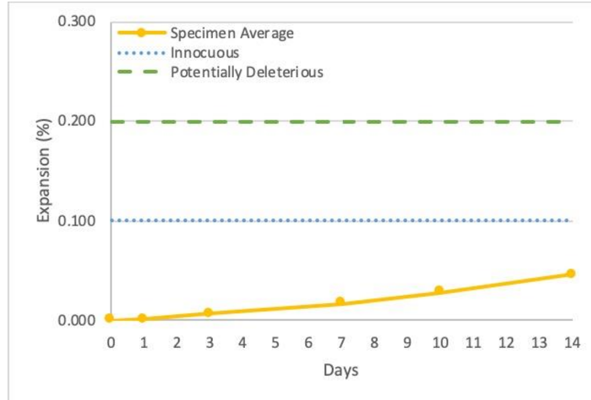
(d) Mix C-4: Specimen Average

Figure 31: ASTM C1260 Results for Mixes C-3 and C-4

The expansion results for the CRH limestone are shown in Figure 32 below. The coarse limestone remained innocuous throughout the 14-day testing period. Thus, the aggregate was considered non-reactive and was used in the subsequent AASHTO T380 tests.



(a) Mix C-NR-1: All Specimens



(b) Mix C-NR-1: Specimen Average

Figure 32: ASTM C1260 Results for Coarse Limestone

5.3 AASHTO T380

This section discusses the procedures used during and the results from the AASHTO T380 (the Miniature Concrete Prism Test) tests conducted on the eight aggregates presented in Section 5.1.

5.3.1 Test Setup

Before beginning the MCPT test procedures, the dry-rodded unit weights of the coarse aggregates were determined in accordance with ASTM C29. This information, in addition to the specific gravity and absorption data acquired in Phase 1 of testing, was necessary to determine the proper mix proportions. Upon completion of the ASTM C29 test, each aggregate was minimally crushed, sieved and recombined to achieve the gradation requirements indicated in Table 10.

Table 10: AASHTO T380 Gradation Requirements [47]

Sieve Size		Mass, %
Passing	Retained on	
12.5 mm (1/2 in.)	9.5 mm (3/8 in.)	57.5
9.5 mm (3/8 in.)	4.75 mm (No. 4)	42.5

The mix design was developed using ACI's Absolute Volume Method and AASHTO T380's prescribed concrete mixture requirements, listed in Table 11. This included adding sodium hydroxide pellets to the distilled mix water to increase the alkali content of the concrete to 1.25% by mass of cement. The material proportions for each MCPT mix can be seen in Table 12.

Table 11: AASHTO T380 Mix Requirements [47]

Variable	Amount
Cement Content kg/m^3 (lb/yd^3)	420 (708)
Coarse Aggregate Volume Fraction	0.65
w/c Ratio	0.45

Table 12: AASHTO T380 Mix Proportions Per ft^3

Mix ID	w/c Ratio	Reagent Water (lb)	Portland Cement (lb)	NaOH Admixture (lb)	Coarse Aggregate (lb)	Fine Aggregate (lb)
F-1	0.45	14.80	26.22	0.099	67.24	67.24
F-2	0.45	14.25	26.22	0.099	67.24	67.24
F-3	0.45	14.28	26.22	0.099	67.24	67.24
F-4	0.45	14.37	26.22	0.099	67.24	67.24
C-1	0.45	13.96	26.22	0.099	65.32	65.32
C-2	0.45	13.41	26.22	0.099	66.96	66.96
C-3	0.45	13.33	26.22	0.099	67.08	67.08
C-4	0.45	13.35	26.22	0.099	66.54	66.54

In the MCPT, the test specimens had dimensions of 50 mm x 50 mm x 285 mm (2 in. x 2 in. x 11.25 in.). The increased mortar bar mold size allowed for minimal crushing of the aggregates. Metal gauge studs were also used in the AASHTO T380 tests. The mortar bar molds used in this research can be seen in Figure 33 below.

Figure 33: AASHTO T380 Mortar Bar Molds

The mortar for each aggregate was mixed in accordance with ASTM C192. Each mix design produced enough concrete for three specimens and a slump test per aggregate. A sample slump test can be seen in Figure 34a below; this particular mix yielded a slump of 1½ inches. The filled molds were placed in the moist room to cure for a period of 24 hours. After demolding, an initial reading was taken and the specimens (Figure 34b) were placed in a water bath at 60°C (140°F) for an additional 24 hours. Following the water bath, a zero reading was taken and the specimens were placed in an approved container and immersed in the 1N NaOH solution at 60°C (140°F) for a period of 56-days. Subsequent expansion measurements were taken at 1, 3, 7, 10, 14, 21, 28, 42 and 56 days.

(a) Slump Test

(b) Specimen in Comparator

Figure 34: Slump and AASHTO T380 Specimen in Comparator

5.3.2 Standard Expansion Limits

AASHTO T380 provides the following expansion limits to assist with interpretation of the test results, shown in Table 11. Specimens that experience an average expansion less than 0.030% at the end of the 56-day testing period are deemed innocuous or non-reactive, while those exceeding 0.24% are considered very highly reactive. The other degrees of reactivity are listed in Table 11.

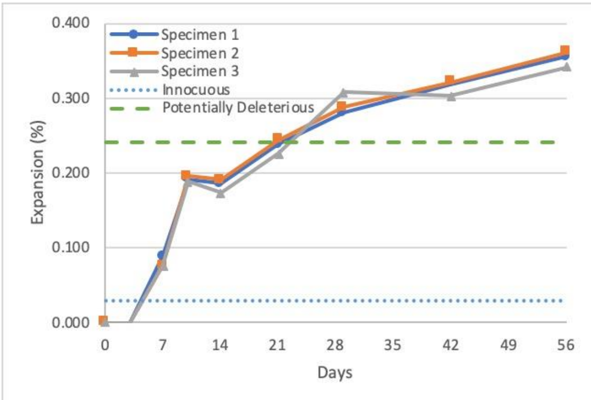
Table 13: AASHTO T380 Expansion Limits [47]

Degree of Reactivity	% Expansion at 56 Days (8 Weeks)	Average Rate of Expansion from (8-12 Weeks)
Non-Reactive	<= 0.030	N/A
Non-Reactive	0.031-0.040	<= 0.010 per two weeks
Low/Slow Reactive	0.031-0.040	> 0.010 per two weeks
Moderate Reactive	0.040-0.120	N/A
Highly Reactive	0.121-0.240	N/A
Very Highly Reactive	> 0.240	N/A

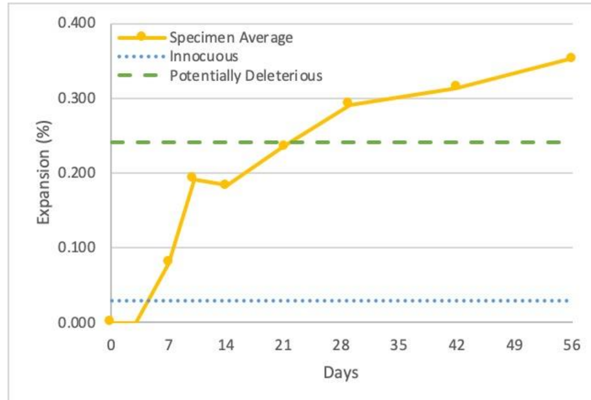
5.3.3 Results from AASHTO T380

The percent expansion versus time is plotted in Figure 35-Figure 38 for the AASHTO T380 tests. The plots on the left show the expansion results for all three specimens across a specific mix, while the plots on the right show the average specimen expansion for that specific mix design. The dotted blue line shows the expansion limit for non-reactive behavior (0.030%). Expansion values exceeding the green dashed line (0.240%) at the end of the 56-day testing period are considered very highly reactive. Values between the dotted/dashed lines can be considered anywhere from non-reactive to highly reactive (refer to Table 12).

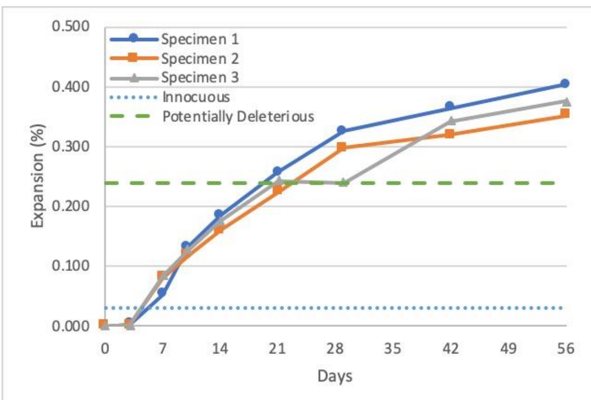
Figure 35 and Figure 36 provide the expansion results for the fine aggregates tested according to AASHTO T380. As can be seen from these graphs, all four of the fine aggregates exceeded the 0.24% expansion limit and demonstrated reactive behavior.



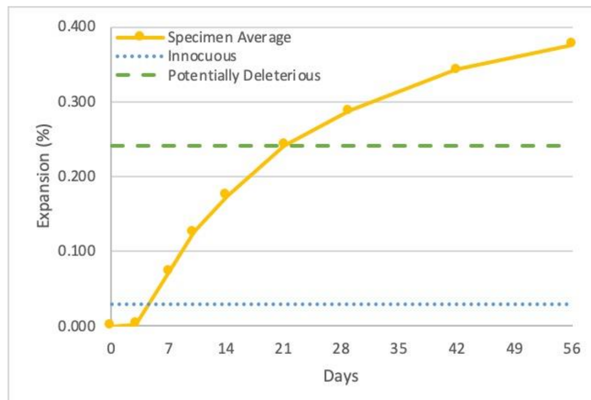
(a) Mix F-1: All Specimens



(b) Mix F-1 Specimen Average

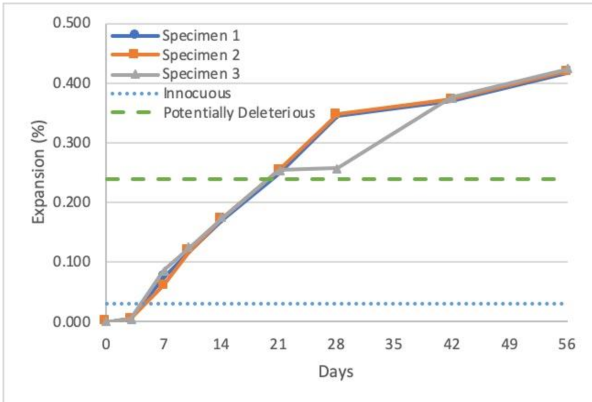


(c) Mix F-2: All Specimens

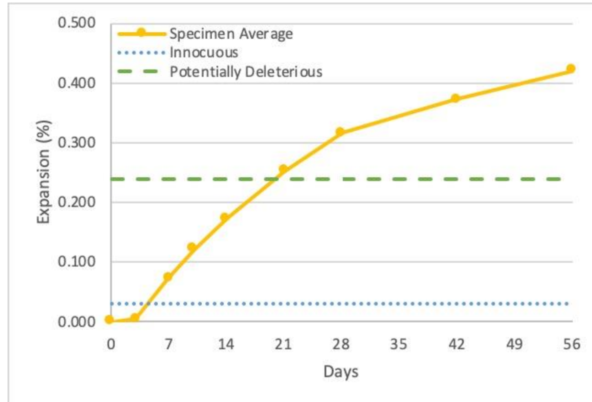


(d) Mix F-2: Specimen Average

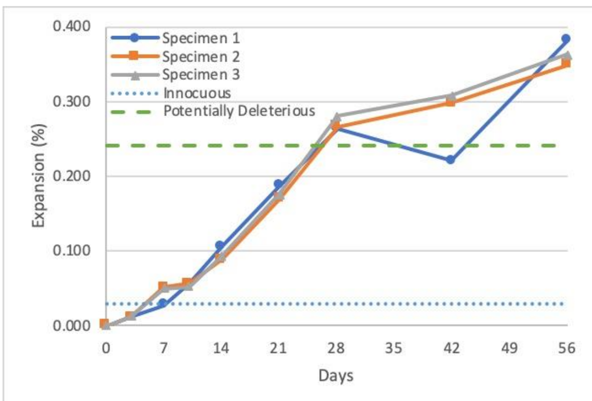
Figure 35: AASHTO T380 Results for Mixes F-1 and F-2



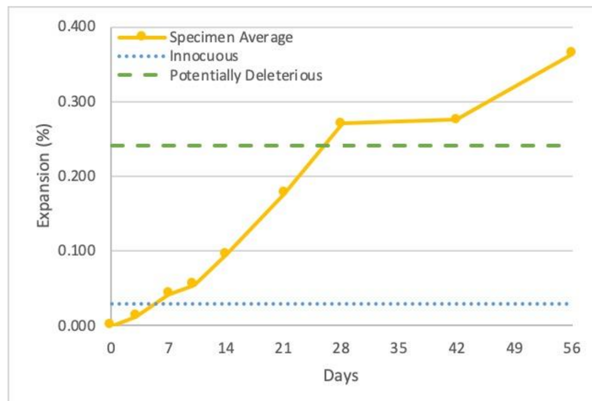
(a) Mix F-3: All Specimens



(b) Mix F-3: Specimen Average



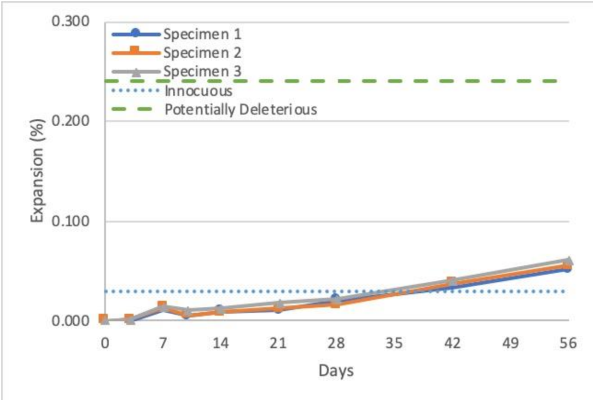
(c) Mix F-4: All Specimens



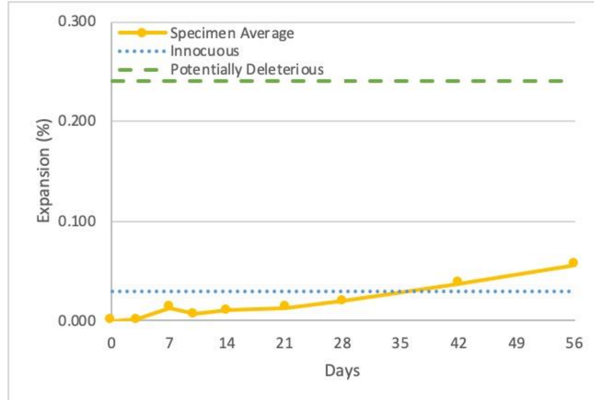
(d) Mix F-4: Specimen Average

Figure 36: AASHTO T380 Results for Mixes F-3 and F-4

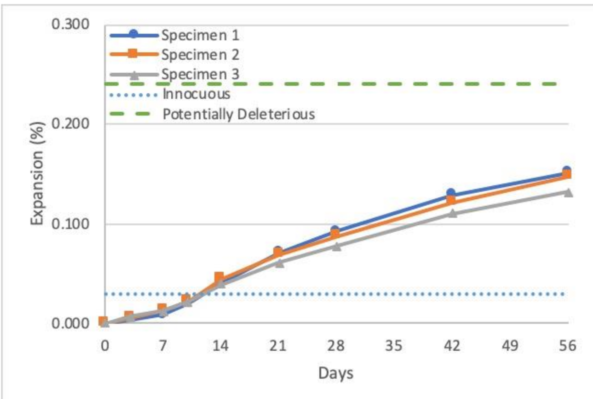
Figure 37 and Figure 38 provide the expansion results for all of the coarse aggregates tested according to AASHTO T380. According to the graphs and expansion limit data, each coarse aggregate exhibited reactive behavior to varying degrees. Aggregate C-1 was moderately reactive, with a 0.056% expansion value at the end of the 56-day testing period. However, it should be noted that aggregate C-1 was considered non-reactive until the 42-day test. Aggregates C-2, C-3 and C-4 were all considered highly reactive, with percent expansion values ranging from 0.121-0.240.



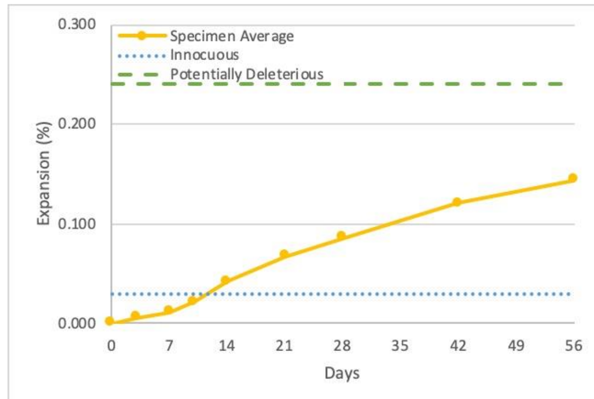
a) Mix C-1: All Specimens



b) Mix C-1: Specimen Average

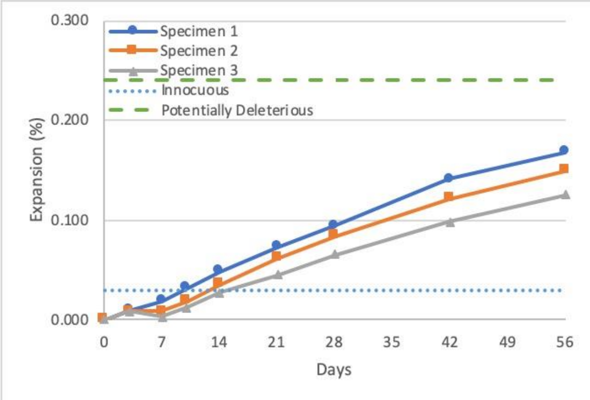


c) Mix C-2: All Specimens

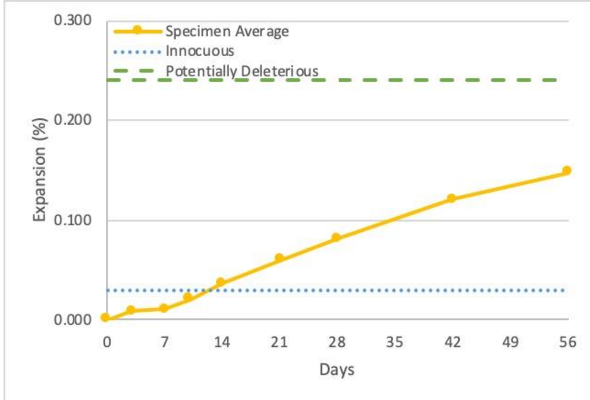


d) Mix C-2: Specimen Average

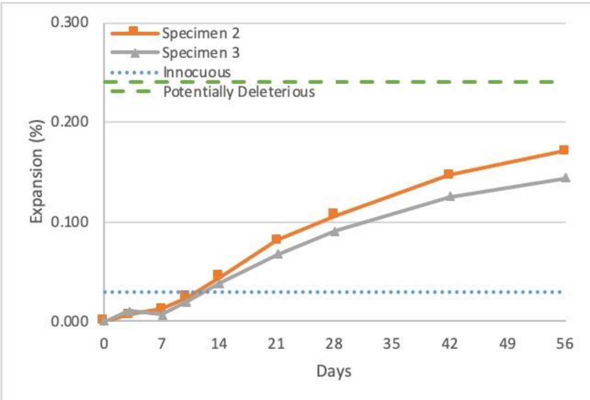
Figure 37: AASHTO T380 Results for Mixes C-1 and C-2



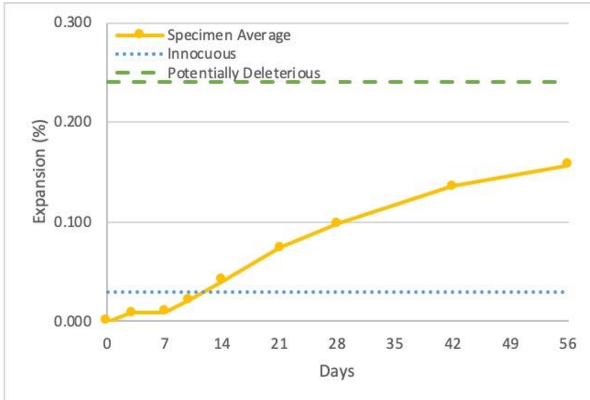
a) Mix C-3: All Specimens



b) Mix C-3: Specimen Average



c) Mix C-4: All Specimens



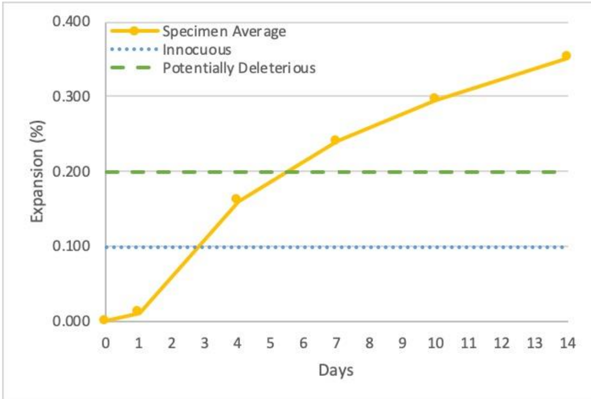
d) Mix C-4: Specimen Average

Figure 38: AASHTO T380 Results for Mixes C-3 and C-4

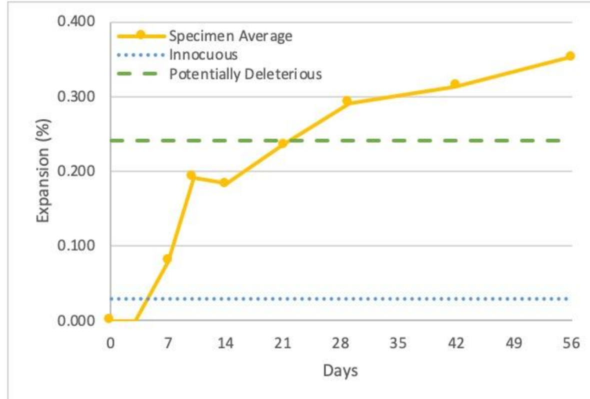
5.4 Comparison Between the Two Methods

The average expansion from the ASTM C1260 and AASHTO T380 tests are plotted in Figure 39-Figure 42. The plots on the left show the average specimen expansion results according to ASTM C1260 for a specific mix, while the plots on the right show the average specimen expansion according to AASHTO T380 for that same mix design. The side-by-side comparison allows for a correlation to be drawn between the ASTM C1260 and AASHTO T380 test results for the same mix design.

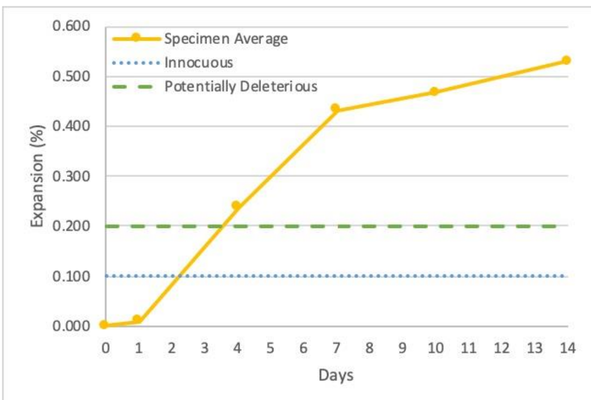
The average results for each fine aggregate are shown in Figure 39 and Figure 40 below. All four of the fine aggregates demonstrated reactive behavior regardless of the test method used. The AMBT results indicated reactive aggregate performance with expansions exceeding the limit of 0.20%. Similarly, the MCPT results revealed that each fine aggregate was very highly reactive and surpassed the expansion limit of 0.24%.



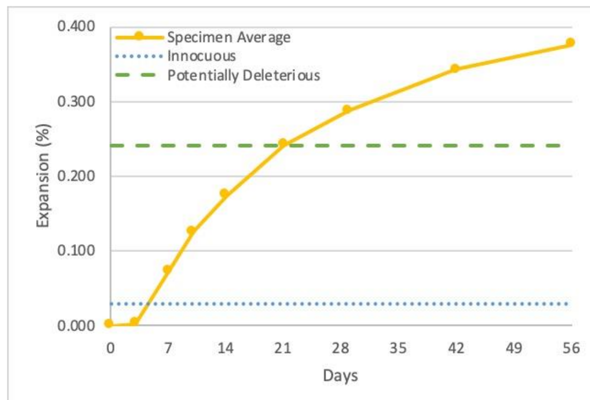
(a) Mix F-1: ASTM C1260 Average



(b) Mix F-1: AASHTO T380 Average

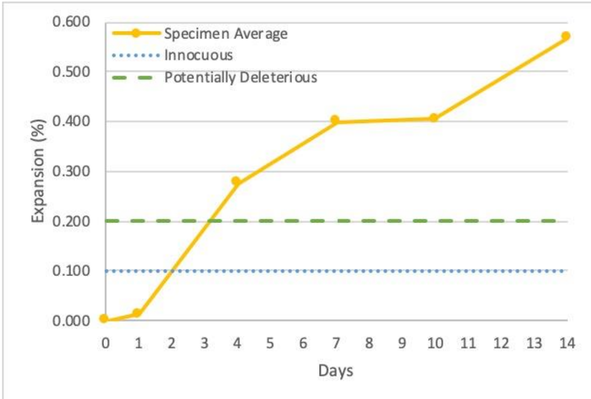


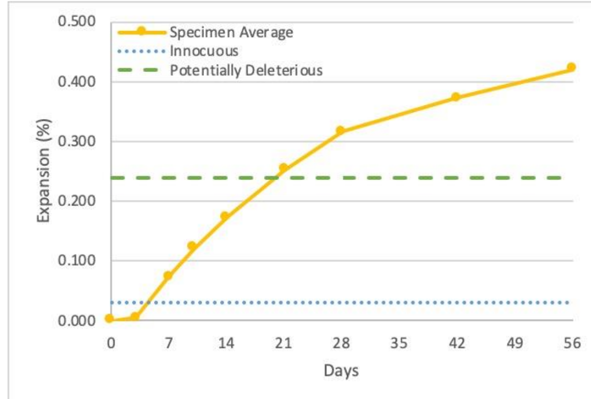
(c) Mix F-2: ASTM C1260 Average



(d) Mix F-2: AASHTO T380 Average

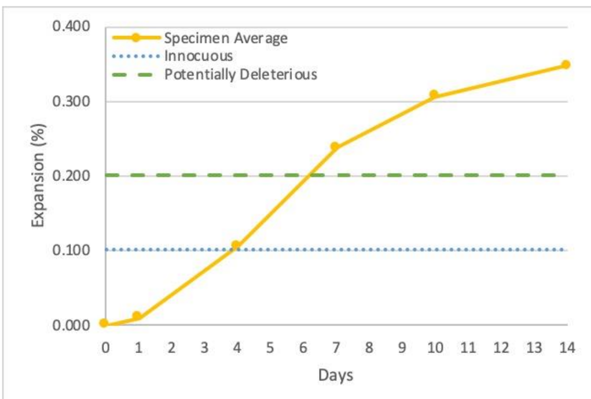
Figure 39: ASTM C1260 vs AASHTO T380 F-1 and F-2 Average Results

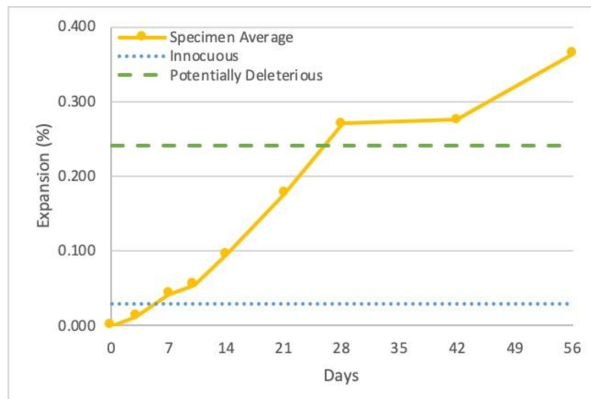




(a) Mix F-3: ASTM C1260 Average

(b) Mix F-3: AASHTO T380 Average



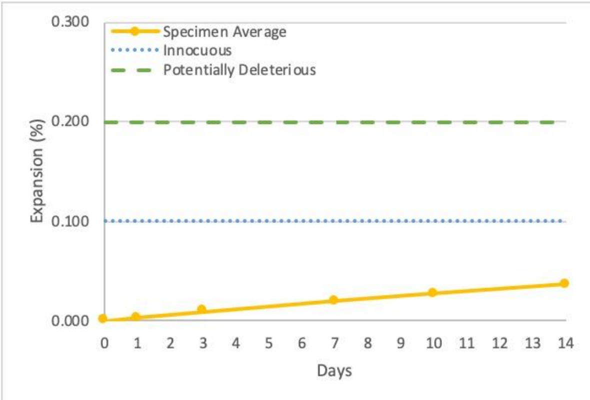


(c) Mix F-4: ASTM C1260 Average

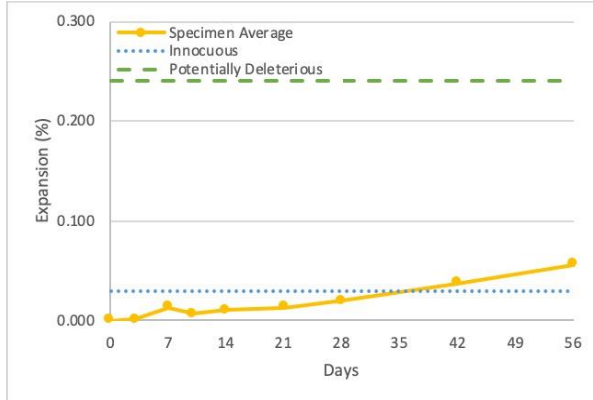
(d) Mix F-4: AASHTO T380 Average

Figure 40: ASTM C1260 vs AASHTO T380 F-3 and F-4 Average Results

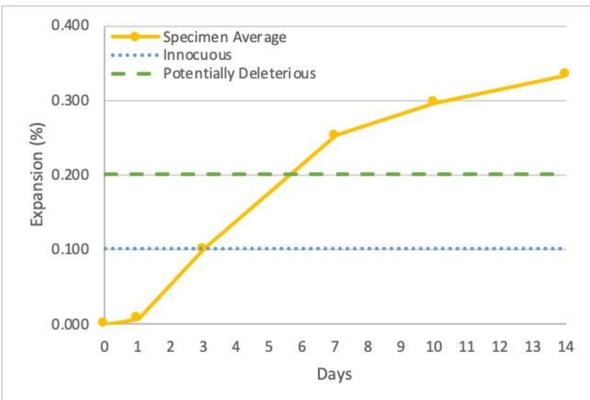
The AMBT and MCPT average expansion results for each coarse aggregate are shown in Figure 41 and Figure 42 below. As can be seen from these graphs, there are discrepancies between the average results of the two test methods. Specifically, coarse aggregate C-1 yielded innocuous or non-reactive results for the ASTM C1260 tests, whereas the AASHTO T380 results indicate moderately reactive behavior for the same aggregate. Aggregates C-2 and C-3 were deemed reactive by ASTM C1260 standards and highly reactive according to AASHTO T380. Finally, aggregate C-4 yielded innocuous results for the AMBT and highly reactive results for the MCPT. Thus, no positive correlation can be drawn between the two test methods for the coarse aggregate testing.



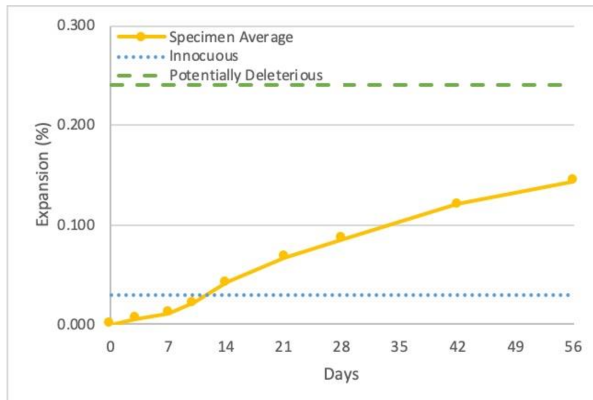
(a) Mix C-1: ASTM C1260 Average



(b) Mix C-1: AASHTO T380 Average

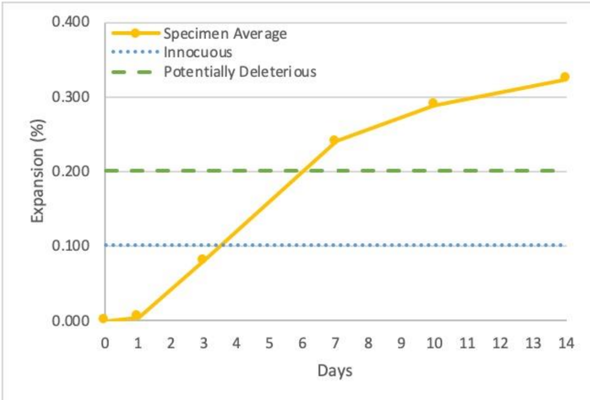


(c) Mix C-2: ASTM C1260 Average

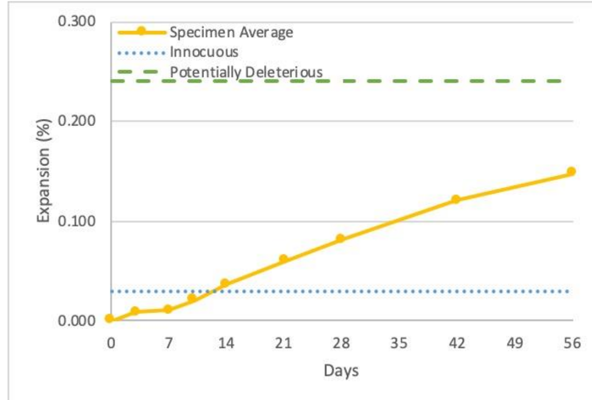


(d) Mix C-2: AASHTO T380 Average

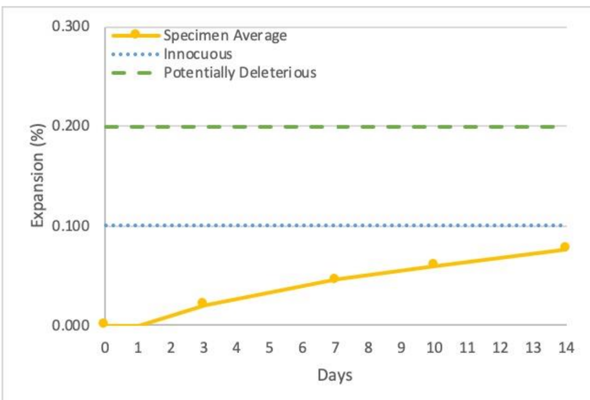
Figure 41: ASTM C1260 vs AASHTO T380 C-1 and C-2 Average Results



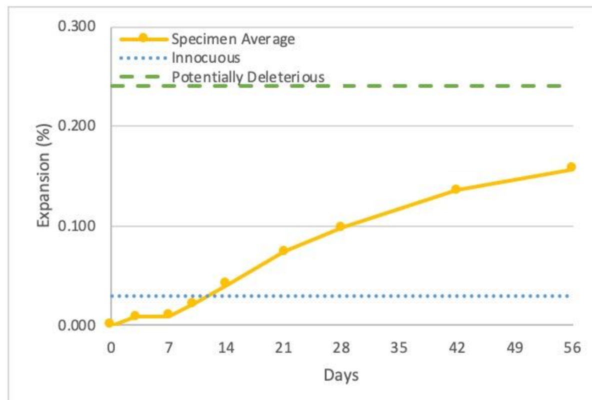
(a) Mix C-3: ASTM C1260 Average



(b) Mix C-3: AASHTO T380 Average



(c) Mix C-4: ASTM C1260 Average



(d) Mix C-4: AASHTO T380 Average

Figure 42: ASTM C1260 vs AASHTO T380 C-3 and C-4 Average Results

Table 12 summarizes the expansion results for both methodologies. According to the ASTM C1260, all of the fine aggregates were reactive and two of the four coarse aggregates were reactive. The AASHTO T380 results indicated that all of the aggregates were reactive to varying degrees.

Table 14: Summary of Expansion Results

Mix ID	Location	ASTM C1260, 14 Days		AASHTO T380, 56 Days	
		Expansion (%)	Degree of Reactivity	Expansion (%)	Degree of Reactivity
F-1	Helena Sand & Gravel - Lake Helena Drive Pit	0.351	Reactive	0.352	Very Highly Reactive
F-2	Knife River Billings - Sindelar Pit	0.529	Reactive	0.376	Very Highly Reactive
F-3	Knife River Billings - Sindelar Pit	0.568	Reactive	0.419	Very Highly Reactive
F-4	Knife River Missoula - Allen Pit	0.347	Reactive	0.364	Very Highly Reactive
C-1	Helena Sand & Gravel - Lake Helena Drive Pit	0.036	Innocuous	0.056	Moderate Reactive
C-2	Knife River Billings - Sindelar Pit	0.333	Reactive	0.143	Highly Reactive
C-3	Knife River Billings - Sindelar Pit	0.324	Reactive	0.147	Highly Reactive
C-4	Knife River Missoula - Allen Pit	0.076	Innocuous	0.157	Highly Reactive

6 SUMMARY AND CONCLUSIONS

6.1 Summary

The primary objective of this research was to evaluate the potential for deleterious ASR in the state of Montana. In connection with this goal, (1) a literature review was conducted to summarize the ASR practices used by neighboring state departments of transportation, as well as several federal agencies, (2) potential cases of ASR damage in the state were identified and investigated, and (3) existing testing methods were used to test the reactivity of several aggregate sources throughout Montana.

In regards to identifying cases of ASR damage in existing concrete, four sites were examined, including two at the Billings Logan International Airport, one at the Willow Creek Dam, and one in an MDT bridge over Belt Creek on US-87/US-89/MT 200. Both the Los Alamos Staining Method (on one site) and petrographic analysis (ASTM C856) were used to characterize the hardened concrete and determine the extent of the ASR damage. The Los Alamos Staining Method offered qualitative results of a highly subjective nature, and although the test results were not entirely conclusive, visible evidence of ASR gel deposits could be seen in both sites' cores. The petrographic analyses were significantly more informative regarding the nature of the concrete constituents and extent of ASR-related deterioration. The results of each case were examined to fully assess the presence/potential of ASR in Montana.

With respect to the aggregate testing methods, the two identified as the most suitable for use by MDT were the Accelerated Mortar Bar Test (ASTM C1260 and the Miniature Concrete Prism Test - AASHTO T380). ASTM C1260 was selected for its timely delivery of results and wide acceptance as an industry standard. AASHTO T380 offered several advantages including its reliability over ASTM C1260, and the minimal testing period (56-days) compared to that required for ASTM C1293 (365-days). A total of eight aggregates were selected from various locations around the state and tested in accordance with ASTM C1260 and AASHTO T380. The results of each testing method were compared to fully characterize each aggregates potential for reactive behavior and to determine if a positive correlation could be drawn between the methodologies.

6.2 Conclusions

Based on this investigation, the following conclusions can be made.

- All of the states and provinces investigated in this research (sans North Dakota) directly address ASR in their material specifications, to varying degrees. The FHWA defers to individual states to determine ASR practices, while the FAA has fairly stringent specifications.
- The results of the Billings Logan International Airport Los Alamos Staining tests indicated the presence of ASR in two sites. The petrographic analysis of the cores from these two sites indicated both the fine and coarse aggregates contained reactive constituents including rhyolite, granite rocks, quartzite, and chert with minimal amounts andesite and basalt. The paste in Site 1 contained only hydrated portland cement, while the paste in Site 2 contained hydrated cement and fly ash. No additional supplementary cementitious materials were detected in either site. Based on the analyses, the ASR damage in both sites was classified as severe according to the DRP scale and Type V according to the scale of Katayama.

- The results from the US-87/US-89/MT 200 bridge indicated the presence of deleterious ASR in the bridge backwall. The paste in this concrete was determined to be composed of hydrated portland cement with no fly ash or other supplemental cementitious materials. The coarse aggregate was natural gravel composed of siliceous and carbonate rocks that include limestone, siliceous limestone, quartzite, chert, and siliceous volcanic rocks. The fine aggregates were natural sand that consisted of rocks similar to those in the coarse aggregate. The findings from this analysis indicate that the concrete shows moderate to severe damage from ASR based on a qualitative scale from DRP, and the progression of the reaction is at Stage IV of six stages using the scale of Katayama.
- The results of the Willow Creek Dam Spillway petrographic examination indicated the coarse aggregate was a natural siliceous gravel, with components including felsic igneous and metamorphic rocks, quartzite, chert and sandstone with minimal amounts carbonate. The paste in the spillway was comprised solely of portland cement. No fly ash, slag cement or other supplementary cementitious materials were identified. The severity of the ASR distress observed in the spillway varied based on location; for example, advanced ASR was detected in the ogee, while other areas of the slab exhibited ASR-related distress to a lesser extent.
- The results of the aggregate testing portion of this research indicated that all the fine aggregates were reactive and very highly reactive according to ASTM C1260 and AASHTO T380, respectively. Two of the four coarse aggregates were innocuous according to ASTM C1260. The reactive aggregates were identical (blind duplicates) and were from the Knife River Billings – Sindelar Pit. Conversely, all of the coarse aggregates were reactive, to varying degrees, according to AASHTO T380.

Overall, while there is not an overwhelming amount of evidence of ASR being a major problem in Montana, this research clearly demonstrated the potential for deleterious ASR in the state. Several sites around the state showed distress from ASR, and all of the tested aggregates showed some reactivity (either with ASTMC 1260 or AASHTO T380). Further, the potential for ASR in Montana may increase as less expensive cements from newer cement plants become more readily available. Cements from these newer plants typically have higher alkali contents than the older plants currently supplying cement in Montana. Further, the cement industry has been trending towards limestone cements and the availability of low alkali cements has been decreasing. It should also be noted that at this time Class F fly ash is commonly used in Montana concrete mixes, which mitigates the potential for ASR. However, Class F fly ash is becoming increasingly more difficult to obtain as the country moves away from coal-fired power plants.

6.3 Recommendations

Based on the findings from this research, the following recommendations are made.

- MDT should not use the Los Alamos Staining method for determining the presence/severity of ASR in existing concrete. This methodology was found to be subjective, with inconclusive results.
- MDT should consider adopting the AASHTO T380 - miniature concrete prism test for aggregate testing when applicable. Previous research has clearly demonstrated the added benefits of this methodology; it provides more accurate, less conservative results than the ASTM C1260 methodology, in significantly less time than the ASTM C1293 methodology. Further, the miniature

concrete prism test can be conducted with the same equipment used for the C1260 (less the forms), requiring a small upfront commitment to make the change.

- The current practice in Montana (limiting the alkalis in cement) seems appropriate/effective for mitigating ASR in Montana and should be continued. It should be noted that the current cement alkali loading limits prescribed by MDT are similar to the limits prescribed for Prevention Level X by AASHTO R80 (Standard Practice for Determining the Reactivity of Concrete Aggregates and Selecting Appropriate Measures for Preventing Deleterious Expansion in New Concrete Construction). However, if the availability of low alkali cements becomes problematic MDT should revisit this approach and consider adopting the methodology prescribed by AASHTO R80 or at least some aspects of this methodlolgy (e.g., prescriptive total alkali loading limits).

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APPENDIX A –ASR MEMO TO MONTANA CONCRETE INDUSTRY

June, 2019

Identifying Alkali Silica Reactivity in Montana

Dear Montana Concrete Industry:

Montana State University (MSU) is currently working with the Montana Department of Transportation and the Montana Contractors Association to evaluate the potential for Alkali Silica Reactivity (ASR) in the state of Montana. Alkali-Silica Reactivity (ASR) is a deleterious reaction that takes place in concrete between alkalis (present in the cement and supplementary cementitious materials) and reactive forms of silica in the aggregates. In the presence of moisture, an expansive gel can form on the surface of the reactive aggregates, resulting in swelling and ultimately a detrimental map cracking pattern similar to that shown in the images that follow.

The overall scope of this research project involves (1) examining state of the practice methods for identifying potential reactive aggregates, (2) establishing recommended aggregate testing methods for future use in Montana, and (3) identifying and documenting cases of ASR damage in the state. Work on the first two items is nearing completion, and work on the third task is just getting underway.

The task of identifying ASR in the state will involve identifying potential sites where ASR may be present, and then extracting samples from these sites for testing to confirm the presence/severity of ASR (e.g., Los Alamos staining method and/or petrographic analysis). Once identified, the test results and key aspects of the concrete will be documented (e.g., location, aggregate source, average humidity and temperature, exposure, etc.).

To help facilitate this last task, MSU is reaching out to the Montana concrete industry (e.g., batch plant operators, contractors, concrete testing laboratories, and public agencies) in hopes of identifying potential sites where concrete may have been affected by ASR. If you have seen signs of ASR damage (see images that follow, and attached document), we ask that you please email and/or call us (see contact information below) and let us know. Also, we would love to hear from you if you have any questions or concerns.

Your help will be greatly appreciated, and will contribute to improving the longevity of Montana concrete.

Sincerely,

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Examples of ASR Damage