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Development of a Field Exposure Site for Predicting and Mitigating ASR

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Final Report

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16. Abstract

Alkali-silica reaction (ASR) is an expansive reaction between the alkalis in the cement and reactive silica in the aggregates. Arkansas is currently witnessing the detrimental effects of ASR at various locations in Arkansas. There has been a significant amount of research conducted using accelerated mortar bar tests (ASTM C1260 and AASHTO T303) and concrete prism tests (ASTM C1293) to determine if specific concrete mixtures show potential for alkali-silica reactivity. More recently, however, outdoor field exposure sites have become popular methods of predicting ASR damage. These sites represent field conditions much better than ASTM C1293 and thus become better predictors for finding reactive aggregates. For this research project, eighteen coarse and three fine aggregates available in Arkansas were collected and tested. The reactivity of these materials were measured using the two standard laboratory methods, the Accelerated Mortar Bar Test (AMBT) ASTM C1260 and the Concrete Prism Test (CPT) ASTM C1293. Exposure block tests, deemed to reflect realistic field results, were also conducted.

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ABSTRACT

Alkali-silica reaction (ASR) is an expansive reaction between the alkalis in the cement and reactive silica in the aggregates. Arkansas is currently witnessing the detrimental effects of ASR at various locations in Arkansas. A petrographic analysis of samples from select locations has confirmed that ASR is present in these areas. Therefore, prevention and mitigation measures must be developed. There has been a significant amount of research conducted using accelerated mortar bar tests (ASTM C1260 and AASHTO T303) and concrete prism tests (ASTM C1293) to determine if specific concrete mixtures show potential for alkali-silica reactivity. The ASTM C1293 test is the preferred method of assessment in laboratories because it best represents field conditions. More recently, however, outdoor field exposure sites have become popular methods of predicting ASR damage. These sites represent field conditions much better than ASTM C1293 and thus become better predictors for finding reactive aggregates. The objective of this research is to identify the potential of alkali-silica reaction in concrete cast with aggregates, both coarse and fine, that are commonly used in Arkansas. For this research project, eighteen coarse and three fine aggregates available in Arkansas were collected and tested. The reactivity of these materials were measured using the two standard laboratory methods, the Accelerated Mortar Bar Test (AMBT) ASTM C1260 and the Concrete Prism Test (CPT) ASTM C1293. Exposure block tests, deemed to reflect realistic field results, were also conducted. Expansion results from these three methods were then used to identify potentially reactive aggregates.

Table of Contents

List of Figures

List of Tables

CHAPTER 1: INTRODUCTION AND RESEARCH OBJECTIVES

1.1 Introduction

Alkali-silica reaction (ASR) is a form of concrete deterioration initiated when the hydroxyl ions in the pore solution and siliceous component in some aggregates form an expansive gel. In the presence of moisture this gel expands, generating an internal strain that creates cracks in the cement paste. The ASR expansion mechanism is driven by the alkalis present in portland cement, siliceous content present in the aggregates, and sufficient moisture to fuel the expansion of the ASR gel (Stanton 1940). Much of the ASR related research focuses on methods to prevent its occurrence, and this involves eliminating any one of these three inputs; alkalis, reactive silica, or moisture.

In 2006, the Federal Highway Association (FWHA) initiated the FHWA ASR Development and Deployment Program. This program was started to assist State transportation agencies to address ASR. FHWA provides tools, guidelines, and protocols to aid in recognizing ASR in the field, protocols for the prevention, diagnosis, and repair of ASR. In recent years the FHWA has funded long-term ASR behavior research to classify and identify the potential for ASR (Thomas et al. 2008; Thomas et al. 2012). The FWHA has also funded research geared toward mitigation of ASR in transportation structures (Fournier et al. 2010; Folliard et al. 2003; Thomas et al. 2007). It is clear that the need to prevent ASR has generated an urgency within the engineering community.

1.2 Research Objectives

The objective of this research is to identify the potential of alkali-silica reaction in concrete cast with aggregates, both coarse and fine, that are commonly used in Arkansas. Results gathered may be used to prevent/reduce alkali-silica reactivity in future construction projects, thus increasing the service life of structures. The results can also provide guidance on the use mitigating materials, such as supplementary cementing materials (SCMs) that can be used to prevent ASR when certain aggregates are used.

1.3 Research Program

For this research project, eighteen coarse and three fine aggregates available in Arkansas were collected and tested. The reactivity of these materials were measured using the two standard laboratory methods, the Accelerated Mortar Bar Test (AMBT) ASTM C1260 and the Concrete Prism Test (CPT) ASTM C1293. Exposure block tests, deemed to reflect realistic field results, were also conducted. Expansion results from these three methods were then used to identify potentially reactive aggregates.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction and Discovery

Stanton (1940) was the first to investigate ASR after several concrete structures in California developed premature deterioration, which was not attributed to any known durability issue at the time. Stanton (1940) determined that mortar bars, which contained high alkali cement and siliceous aggregates, produced deleterious expansion when exposed to damp conditions for over one year (Stanton 1940, and ACI 1998). In addition Stanton's work provided the first test method to evaluate the expansive reaction between cement and aggregates (ACI 1998). Stanton (1940) provided recommendations to prevent ASR, such as limiting the cement alkali content to 0.6 percent and/or replacing a portion of the cement with a supplementary cementitious material (SCM) (ACI 1998).

2.2 Alkali-Silica Reaction (ASR)

When reactive siliceous minerals, from the aggregate, react with highly alkaline cement pore solution, ASR occurs (ACI 1998, and Diamond 1989). The presence of sodium (Na), potassium (K), and other alkalis within the pore solution produces a very high equilibrium pH (Diamond 1989). Reactive silica minerals dissolve when exposed to the high pH cement pore solution, and when sufficient calcium hydroxide is available, expansive alkali-silica gel will precipitate from the dissolved silica and alkali solution (Diamond 1989). Once alkali-silica gel deposits form, the gel will imbibe water from the surrounding cement pore solution and expand (Powers and Steinour, 1955, and Diamond 1989). It is important to note that, depending on the availability of calcium hydroxide within the cement pore solution, not all alkali-silica gels are

expansive (Powers and Steinour, 1955, Diamond 1989, and Helmuth 1993). As the gel continues to absorb water, the gel will produce an expansive pressure on the surrounding cement matrix and aggregates. The pressure is often sufficient to form microcracks within the cement matrix. In addition, as the reactive aggregate particles are eroded, and gel deposits form within the aggregate, the microcracks will extend into the aggregate particles. The micro-cracks will continue to expand until visible cracks develop (Diamond 1989).

Four components are required for ASR and expansion to occur within concrete (ACI 1998, and Stark 1993). The necessary conditions which produce expansive ASR are shown in Figure 2.-1. Reactive silica within the aggregate is necessary for the development of alkali-silica gel. Also, a high concentration of alkalis is necessary to produce the high pH within the cement pore solution. A limited amount of mobile Calcium hydroxide, within the cement pore solution, is required for expansive gels to precipitate (Diamond 1989). Additionally, sufficient moisture is required for alkali-silica gel expansion to occur and continue (ACI 1998).
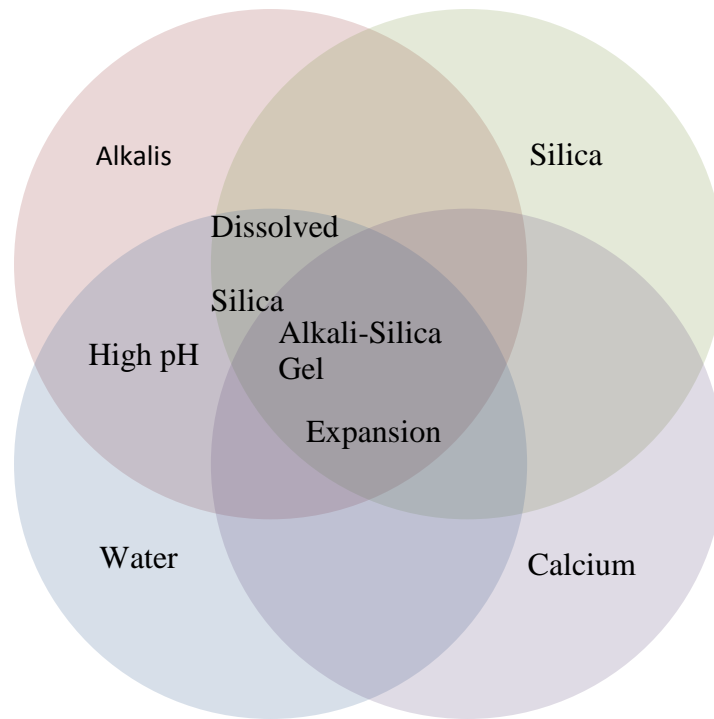
Figure 2.2-1. Components required for alkali-silica gel to develop and expand.

Reactive siliceous minerals are present within many coarse and fine aggregates. Some common siliceous minerals include: opal, chalcedony, quartz, cristobalite, tridymite, volcanic glasses, and chert (ACI 1998, and Helmuth 1993). Reactive siliceous minerals have a disordered crystal structure which may react and dissolve when exposed to a highly alkaline cement pore solution (Powers and Steinour, 1955, Helmuth 1993). However, not all minerals have the same level of disorder within the silica crystals. Therefore, disordered minerals such as opal will react much faster than ordered minerals such as quartz. As the concentration of alkalis within the cement pore solution increases, the silica minerals are dissolved faster and more expansion occurs (Powers and Steinour, 1955, and Helmuth 1993).

2.2.1 Alkalis and Calcium

Soluble alkalis including sodium hydroxide (NaOH) and potassium hydroxide (KOH) are released into the cement pore solution during hydration (Helmuth 1993). As hydration continues some of the alkalis are used up in the reaction products; however, the water is also used up and the alkali concentration increases (Powers and Steinour, 1955, Diamond 1989). Diamond (1989) discovered a linear relationship between alkali concentration in cement pore solution and the available alkali content of the cement (Helmuth 1993). As the available alkalis within the cement increases, the concentration of alkalis within the pore solution of the concrete increases. Powers (1955) stated the importance of calcium in the formation of expansive alkali-silica gels. When excess calcium hydroxide is available within the alkali-silica gel, the gel is unstable and produces dissolved silica which will not expand. However, when the calcium-alkali-silica system produces a barrier to the transport of excess calcium to the reaction site, a stable alkali-silica gel is produced which then absorbs water and expands (Powers and Steinour, 1955, and Helmuth 1993). The process of dissolving silica minerals in the presence of highly alkaline pore solution will continue until either all of the reactive silica or all of the alkalis are consumed (Helmuth 1993).

2.2.2 Expansion and Moisture

ASR is a two phase deleterious reaction (Diamond 1989, and Helmuth 1993). The first phase described above involves the precipitation of expansive alkali-silica gel during the reaction between highly alkaline cement pore solutions and reactive siliceous minerals (Diamond 1989). The second phase involves the absorption of water into the alkali-silica gel, which produces expansion and pressure (Diamond 1989, and Helmuth 1993). The alkali-silica gel is insoluble and acts as a membrane between the gel and cement pore solution (Powers and Steinour, 1955, and

Diamond 1989). Pore solution is drawn, by the reduced free energy within the alkali-silica gel, through this membrane into the alkali-silica gel and produces expansion (Powers and Steinour, 1955, Diamond 1989, and Helmuth 1993). However, the expansion is limited by the availability of moisture to the expansive gel (Stark 1990, and Fournier et. al, 2004). Stark (1990) reported a limiting value of 80 percent internal relative humidity, at a reference temperature of 21 to 24 degrees Celsius, which is required for expansion to continue.

2.3 Diagnosing ASR

The symptoms of ASR are similar to other durability mechanisms which occur in concrete (Fournier et. al, 2004). Some of the visible symptoms of ASR include map cracking at the surface of the concrete, or relative displacement of concrete elements (Stark 1991, Fournier et. al, 2010, ACI 1998, and Fournier et. al, 2004). Additional symptoms include pop-outs, surface discoloration, and/or gel exudation at the surface (Stark 1991, Fournier et. al, 2010, ACI 1998, and Fournier et. al, 2004). Often the concrete is discolored along cracks, especially when the concrete is moist (Stark 1991, Fournier et. al, 2010, and ACI 1998). Some typical symptoms of ASR are shown in Figure 2.3-1. Expansion greater than 0.04 percent will lead to visible cracking in unreinforced concrete (Ideker et. al, 2012a).

Figure 2.3-1. Example of ASR symptoms such as map cracking, cracks parallel to restraint, gel exudation, surface discoloration, etc. (Picture used with permission from Richard Deschenes).

Map cracking occurs in concrete elements which are subjected to cyclic environmental conditions. The concrete within an element will expand more than the outer surface due to a temperature and humidity gradient which develops between the surface and interior concrete (Fournier et. al, 2004). As shown in Figure 2.3-2, expansion of the concrete will produce tensile stresses and micro-cracks in the interior concrete, and drying shrinkage near the surface will cause map cracking in the exposed surfaces of the concrete (ACI 1998, Fournier and Berube, 2000a, and Fournier et. al, 2004).
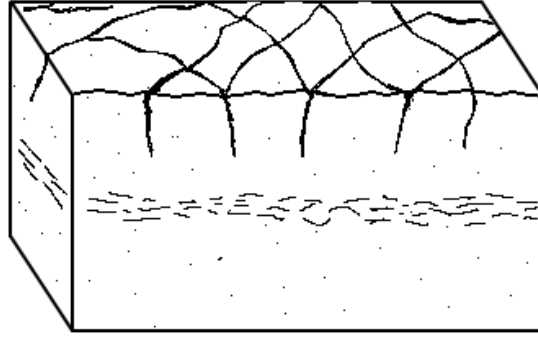
Figure 2.3-2. Concrete element showing map cracking at the surface and microcracks parallel to the surface, due to the humidity gradient between the exposed surface and the internal concrete (Picture recreated from ACI 1998).

Concrete with anisotropic restraint, either by internal reinforcement or adjacent concrete members, will develop cracks parallel to the direction of restraint (Stark 1991, Fournier et. al, 2004). Unfortunately, it is not possible to diagnose ASR with visual symptoms because they are common symptoms of other durability problems such as freezing and thawing, corrosion, sulfate attack, and/or plastic or drying shrinkage (Thomas et. al, 2011, and ACI 1998). Therefore petrographic examination of the concrete is required to diagnose the presence of ASR (Stark 1991, and ACI 1998).

2.3.1   Petrographic Examination

Conclusive evidence of ASR is found in concrete only though petrographic examination. The examination includes both microscopic and macroscopic identification of symptoms of ASR. Macroscopic evidence of ASR includes deposits of alkali-silica gel which is identified visually and chemically (Fournier et. al, 2004, and ACI 1998). Deposits of alkali-silica gel develop within

voids in the cement pore solution and within cracked aggregate particles (Fournier et. al, 2004, and Fournier et. al, 2010). This evidence is shown in Figure 2.3.1-1.
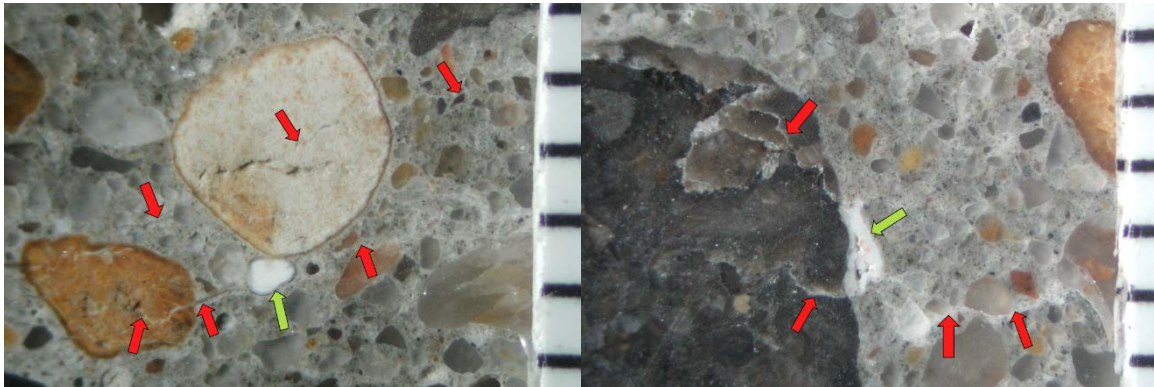
Figure 2.3.1-1. Typical petrographic features, including microcracking within fine aggregate particles and extending into the cement matrix (red arrows) and alkali-silica gel deposits (green arrow). (Picture used with permission from CTLGroup).

Additionally, reaction rims are sometimes present within the interfacial transition zone around reactive aggregate particles (Fournier et. al, 2004, Fournier et. al, 2010, and ACI 1998). Microscopic evidence of ASR may include micro-cracking within aggregates and/or the cement matrix, reaction rims, and alkali-silica gel (Fournier et. al, 2004, and Fournier et. al, 2010). Petrographic examination is necessary to provide a conclusive diagnosis of the presence of ASR and other forms of deterioration that may have occurred as a result of, or in combination to, ASR (Fournier 2004 et. al, ACI 1998, and Fournier et. al, 2010).

2.4 Prevention of ASR

Several options are available for the prevention of ASR in concrete. The most effective method for the prevention of ASR involves controlling one or more of the constituents required

for ASR to develop (ACI 1998). Moisture is required for ASR to develop and expansion to progress; however, limiting water is a prohibitive method of preventing ASR. The most common method of prevention involves limiting the concrete pore solution alkalinity (ACI 1998). Cement alkalis are most readily available within the cement pore solution (Diamond 1989, and Thomas 1995). However, alkalis are sometimes contributed by SCMs or even aggregates (ACI 1998, and Thomas 1995). The use of low alkali cements will reduce cement pore solutions alkalinity, and a limit of 0.6 percent Na_2O_e is recommended when used in combination with reactive aggregates (Thomas et. al, 2006a, ACI 1998, and Stanton 1940). However, in certain highly reactive aggregates this limit is not sufficient (Stark 1980, ACI 1998, and Swamy et. al, 1988b). In addition, SCMs dilute high alkali cements or bind the available alkalis within the hydration products (ACI 1998). Some of the SCMs which prevent or reduce ASR expansion include slag cement, fly ash, calcined clays, rice husk ash, and silica fume (ACI 1998, and Thomas et. al, 2006a). The safe replacement rate of cement with an SCM will depend on the cement alkalis, aggregate reactivity, selected SCM, and chemical and physical composition of the SCM (ACI 1998, Thomas et. al, 2006a, and Thomas 1995).

2.4.1   SCM Mechanisms

The mechanisms by which SCMs prevent ASR depends on the SCM, the compositions of the SCM, and the level of replacement (ACI 1998, and Thomas 1995). The first mechanism involves diluting the alkalis within the cement, by replacing a portion of the cement with a lower alkali SCM (ACI 1998, and Thomas 1995). The second mechanism involves the production of secondary calcium silicate hydrates which bind alkalis and reduce the pore solution alkalinity (ACI

1998, Thomas 1995, and Duchesne and Berube 1994). Additionally, the pozzolanic reaction can consume calcium hydroxide and reduce the expansion of ASR gel (ACI 1998, Thomas 1995, and Duchesne and Berube 1994). Finally, the improved production of calcium silicates produces lower permeability and reduces the mobility of alkalis (ACI 1998, and Thomas 1995).

The efficacy of an SCM also varies with the composition of the particular SCM. The lime content of fly ash depends on the coal used in the production of the fly ash. Class C ashes contain 8 percent or more lime, and are both cementitious and pozzolanic. The cementitious properties of Class C ashes releases additional alkalis and makes Class C ashes less effective at preventing ASR; therefore, a replacement rate of 40 to 50 present is required (ACI 1998, Shehata and Thomas, 2000, and Thomas 1995). Class F fly ashes contain less than 8 percent lime and are purely pozzolanic, the improved production of calcium silicates produces a reduction in ASR expansion with replacement rates between 15 and 30 percent (ACI 1998, and Shehata and Thomas, 2000).

Slag cement is also used to prevent ASR expansion; however, at replacement rates between 25 and 50 percent (ACI 1998, and Thomas 1995). The higher replacement rates are required because the alkalis within the slag cement are released into the pore solution and require additional secondary hydration to bind and dilute the alkalis (ACI 1998, and Thomas 1995). An additional option for the prevention of ASR is lithium nitrate, which is added to concrete mixtures and will prevent the development of expansive alkali-silica gel (Thomas et. al, 2006a, and Folliard et. al, 2007). The required dosage of lithium is determined through accelerated testing (Folliard et. al, 2007).

2.5 Test Methods for Assessing Alkali-Silica Reactivity

Several laboratory test methods exist which can determine the alkali-silica reactive potential of aggregates or aggregates and cement combinations. The first test method was developed by Stanton (1940) and involved producing mortar bars with the selected aggregate and cement (ACI 1998, and Stanton 1943). The mortar bars were stored in moist conditions within sealed containers for up to two years, with expansion measured periodically. The test method was similar to the ASTM C227 mortar-bar test which was developed in 1950 (ACI 1998, and Thomas et. al, 2006a). The mortar bar test method was useful for evaluating reactive aggregates; however, the test had several limitations. The mortar-bar test method was susceptible to alkali leaching due to the transport of alkalis through water which precipitated on the surface of the mortar bar (Stanton 1943, Thomas et. al, 2006a, ACI 1998, and Rogers and Hooton 1991). In addition, the mortar bar test duration was one year and did not accurately access slowly reactive aggregates. Therefore, Davies and Oberholster (1986) developed an accelerated mortar bar test (AMBT) method, based on recommendations originally made by Stanton (1943). The AMBT evaluated the expansion of reactive aggregates in only 16 days. The AMBT could also detect expansion in slowly reactive aggregates, and was not susceptible to alkali leaching due to the additional alkalis provided by the NaOH soak solution (ACI 1998, and Thomas et. al, 2006a).

2.5.1   Accelerated Mortar Bar Test (AMBT)

The AMBT is similar to the mortar-bar test, with a modified storage environment. The mortar bars are one inch square and 11.25 inches long, with stainless steel gage studs attached at the ends for length comparator readings. The aggregate gradation is standardized to improve repeatability. Coarse aggregates are crushed and sieved to match the gradation. However, crushing

the aggregate potentially changes the reactivity of the aggregate (Du-you et. al, 2004). The mortar bars are stored in 1 N NaOH solution to prevent alkali leaching, and at 80 degrees Celsius to accelerate expansion (ACI 1998, and Rogers and Hooton 1991). Davies and Oberholster conducted testing on the mechanism and reaction products of alkali-silica gel within AMBT specimens, and concluded that the mechanism and reaction products were not altered by the accelerated conditions (Davies and Oberholster 1988). The duration of the AMBT is 16 days, which includes one day of curing, an additional day of storage in water at 80 degrees Celsius, and then 14 days stored at 80 degrees Celsius in NaOH (ACI 1998, Thomas et. al, 2006a, Ideker 2012a, and ASTM C1260). Due to the alkali solution used in storage, the cement alkali used for testing has little effect on the final expansion (Du-you 2004).The AMBT was standardized under ASTM C1260 and is used to screen aggregates for potentially deleterious expansion (ASTM C1260).

Due to the harsh environment in which the mortar-bars are stored, the AMBT method often falsely assesses aggregates as reactive which exhibit good field performance in concrete (Thomas et. al, 2006a, Ideker et. al, 2012a, and Touma et. al, 2001). Therefore, additional test methods, such as the concrete prism test (CPT), are conducted for aggregates which produce potentially deleterious expansion during AMBT testing (Thomas 2006a, Ideker et. al, 2012a, and Fournier et. al, 2000b). Although the AMBT has been standardized, there is some disagreement about the final expansion which indicates reactive aggregates. Generally, expansion greater than 0.10 percent at 14 days indicates a potentially expansive aggregate (Fournier et. al, 2000b). However, some authors have recommended lowering this limit to 0.08 percent for slowly reactive aggregates (Stark 1993, and ACI 1998).

The AMBT is also applicable when assessing the safe replacement rate of SCMs for the prevention of ASR in mortars with known reactive aggregates (Fournier et. al, 2000b, and Thomas

et. al, 2006a). A specification was developed for this purpose and standardized as ASTM C1567 (ASTM C1567). The test method is similar to the AMBT, with the exception of the cementitious materials. A portion of the cement is replaced with the selected SCM, and the test duration extended to 28 days (Touma 2000). The test method provides conservative replacement rates for SCMs when preventing ASR in concrete is required. However the test method is not applicable when evaluating safe cement alkali levels (Thomas et. al, 2006a). The two-year CPT is considered more reliable when evaluating reactive aggregates and safe SCM replacement rates (Thomas et. al, 2008).

2.5.2 Concrete Prism Test (CPT)

The AMBT has several limitations which prevent the test from producing reliable results for certain aggregates. In addition, the AMBT is only applicable to evaluate aggregates in mortar samples. Expansion produced in AMBT mortar bars does not correlate to expansion developed in the field (Ideker et. al, 2012a). The concrete prism test (CPT) was developed to more accurately predict the expansion of concrete containing reactive aggregates (ACI 1998, and Ideker et. al, 2012a). The CPT is standardized under ASTM C1293 (ASTM C1293). The CPT specifies the coarse aggregate gradation, cement content, and water to cement ratio (ACI 1998, and ASTM C1293). The cement alkali content is limited to 0.90 ± 0.1 percent. However, the alkalis are boosted to 1.25 percent through the addition of NaOH to the mixing water (ACI 1998, Thomas et. al, 2006a, and ASTM C1293). The test duration is one year, and the specimens are stored in a sealed container over water at a constant temperature of 38 degrees Celsius, and relative humidity greater than 95 percent (ASTM C1293).

The concrete prism test is highly susceptible to alkali leaching and a wicking material is required within the storage container to prevent moisture from precipitating on the surface of the prisms (Rogers and Hooton 1991, Thomas et. al, 2006a, Ideker et. al, 2012a). The limitations of the CPT include the relatively long test duration and the effects of alkali leaching (ACI 1998, and Thomas et. al, 2006a). In addition, because alkalis are boosted to 1.25 percent, the test is not applicable when evaluating the safe cement alkali level of a concrete mixture (Thomas et. al, 2006a). The two-year CPT is the most accurate accelerated method for assessing the potential for deleterious expansion in concrete, and for determining safe replacement rates of SCMs (Ideker et. al, 2012a, Ideker et. al, 2012b, Touma et. al, 2001, and Fournier et. al, 2000b). Fournier et. al (2000b) recommended evaluating the field combination of fine and coarse aggregates together, with a two year test duration.

2.5.3   Outdoor Exposure Block Test

Due to the limitations within accelerated laboratory test methods, field service records are often the best method for determining the long term performance for a concrete mixture (Thomas 2006b, Ideker 2012a, and Ideker 2012b). One method of measuring field performance under controlled conditions is the large block exposure site test. One advantage of the large block test is the ability to evaluate a concrete mixture in field conditions. Additionally, the concrete mixture may contain cements with any alkali level, and any combination of SCMs (Thomas et. al, 2006b, and Ideker et. al, 2012b). Expansion is monitored using detachable mechanical (DEMEC) type strain gages in combination with gage studs affixed to the surface of the blocks. Alternately, vibrating-wire strain gages are embedded within the concrete during placement, with the advantage of continuous expansion monitoring. Internal relative humidity and temperature are monitored

with hand-held instruments and probes, which are inserted into holes, drilled in the block. Several outdoor exposure sites have been constructed in the United Kingdom, Canada, and the United States (Thomas et. al, 2006b, Ideker et. al, 2012b, and Thomas et. al, 2006a).

Although there are no standards available for the large block exposure site test, recommendations are available in the literature. The size of blocks range from 350 mm to 900 mm, although some blocks are as large as 3 m (Ideker et. al, 2012b, and Thomas et. al, 2006a). Researchers at the International Center for Aggregate Research (ICAR) recommend blocks with dimensions of 380 x 380 x 710 mm (Ideker et. al, 2012b, and Folliard et. al, 2012). Cement alkalis are increased to 1.25 percent Na_2O_e when evaluating mitigation measures. However, low alkali cement mixtures are evaluated without increased alkalis (Ideker et. al, 2012b). The coarse aggregate volume fraction is the same as used in the CPT; however, it is also possible to evaluate the natural gradation (Ideker et. al, 2012b, and Folliard et. al, 2012).

The outdoor exposure site test is considered the most accurate method of measuring field performance of concrete mixtures. The test method is often used to validate the results of accelerated laboratory test methods. The ambient conditions vary with the region where the site is constructed, and best represent the conditions which the concrete is subjected to in the field (Ideker et. al, 2012b). The major limitation of the outdoor exposure site test is the extended time required to produce results, which is greater than 10 years when evaluating field performance (Ideker et. al, 2012b). However, the test can produce results on preventative measures and mitigation methods in less time.

2.6 Test Methods for ASR in Existing Concrete Elements

In structures which exhibit ASR symptoms, it is often necessary to monitor the progression of damage and, if applicable, to monitor the effectiveness of mitigation methods (Fournier et. al, 2004). Damage is measured through a combination of in-situ measurements, which may include destructive and/or non-destructive methods. The most common destructive method involves removing core samples for analysis. Core samples are then used for various tests including petrographic analysis, concrete mechanical properties tests, or quantitative damage indices (e.g. Damage Rating Index). An additional test method involves subjecting cores to accelerating conditions (similar to the AMBT or CPT) to determine the potential for future expansion, which is then used in conjunction with in-situ expansion data to evaluate the current state of a concrete element and to develop treatments and retrofits as they become necessary (Fournier et. al, 2004).

Petrographic examination is an important step in diagnosing the presence of ASR and/or other deterioration mechanisms within concrete. Core samples are extracted from the concrete element and then visually inspected for symptoms of ASR. Some symptoms which are identified under the microscope include microcracks, reaction products (ASR gel), reaction rims, and changes in the interfacial transition zone (Fournier et. al, 2004). Before these symptoms are identified, the concrete sample is polished or cut into thin sections and then analyzed under a low powered microscope (Fournier et. al, 2004). Additionally, damage indices are used to quantify and compare the petrographic features of ASR (Fournier et. al, 2004). The damage rating index (DRI) is a quantitative measure of several petrographic features and is useful for comparing the progression of ASR damage in concrete over time, or between separate concrete elements within a structure (Smaoui et. al, 2004, and Shrimer 2000).

Mechanical tests of core samples are useful when evaluating the loss of engineering properties within concrete affected by ASR (Fournier et. al, 2004). Compressive and tensile strength tests of core samples are used to determine the loss of strength within a concrete element due to ASR deterioration (Fournier et. al, 2004). Tensile strength deteriorates much more rapidly than compressive strength during ASR expansion and is a better indicator of the level of damage (Fournier et. al, 2004, and Swamy and Al-Asali, 1988a). The stiffness Damage Index was developed as a method of estimating the expansion-to-date within a concrete element. The test method involves subjecting a core to five cycles of loading and unloading, and measuring the plastic deformation and energy dissipation (Fournier et. al, 2004, and Smaoui et. al, 2004).

A more common test involves subjecting cores to expansion accelerating conditions to estimate the potential for future expansion (Fournier et. al, 2004). Two cores are required for this method, the first core is subjected to the same conditions as the CPT (38 degrees C and 95 percent RH) and the expansion is monitored to assess the degree of future expansion. The second core is submerged in 1N NaOH at 38 degrees Celsius and the expansion monitored to determine the absolute degree of reactivity of the aggregates within the concrete (Fournier et. al, 2004).

The destructive test methods provide some insight into the future expansion and deterioration which will occur in the concrete. These methods in addition to in-situ monitoring are used to develop a plan of mitigation and/or remediation which will extend the useful life of the concrete element.

2.6.1   In-situ (Field) Monitoring

In-situ expansion monitoring is used to determine the expansion rates and potential of concrete structures (Fournier et. al, 2004). Reference gage studs are affixed to the concrete. The

gage studs have a small machined indent which is matched up with the points on a DEMEC gage. Periodic measurements are then taken between two gage studs to monitor the progression of expansion along an axis of the concrete element (Fournier et. al, 2004). Vibrating wire strain gages have also been used successfully to monitor expansion continuously, with the ability to transmit measurements automatically (Fournier et. al, 2004).

Another important characteristic to monitor is the change in internal relative humidity and temperature. Portable humidity probes are available which are inserted into holes drilled into the concrete element, and report the internal temperature and relative humidity (Fournier et. al, 2004). However, the probe must remain in the concrete until equilibrium is reached between the air in the hole and the moisture in the cement pore solution. The internal relative humidity is monitored to determine the effectiveness of a mitigation method. If the internal relative humidity is reduced below 80 percent, ASR will arrest.

2.7 Mitigation of ASR

Although there are several effective methods for the preventing of ASR, concrete structures which develop ASR are still constructed. Mitigating ASR in hardened concrete is much more difficult and expensive than preventing ASR in fresh concrete. Several repair or rehabilitation methods are available for various concrete structures (ACI 1998, and Fournier et. al, 2004). Transportation structures such as pavements, bridge decks and elements, and median barriers often have large surface areas compared to volume. In structures with relatively thin cross sections controlling internal moisture is often the best method of mitigating ASR (ACI 1998, Fournier et. al, 2004). As stated earlier, an internal relative humidity greater than 80 percent will cause expansion to continue (ACI 1998, and Stark 1990). In some concrete elements protection from

rain or groundwater is possible, and will reduce expansion. The most promising method of moisture protection for bridge elements and median barriers has been silane (ACI 1998, Berube et. al, 2002b, Drimalas et. al, 2012, Thomas et. al, 2012, and Fournier et. al, 2004). However, there is very little published literature on the long term efficacy of silane on concrete pavements.

Abundant literature has been published on the ability of topical lithium treatment used to mitigate ASR. If a sufficient dose of lithium is introduced into the concrete pore solution, ASR expansion will cease (Folliard et. al, 2007, and Stark et. al, 1993). However, due to the low permeability of hardened concrete, it is difficult to introduce the required dose of lithium through the full depth of a concrete element (Johnston et. al, 2000, Folliard et. al, 2012, and Tuan et. al, 2005). Several methods have been used to increase the penetration of lithium into concrete structures, with limited success (Stokes et. al, 2003, Thomas et. al, 2012, and Drimalas et. al, 2012). Concrete elements treated topically, through vacuum impregnation, or electrochemical impregnation with lithium nitrate produced varying levels of penetration depending on the level of concrete cracking at the time of treatment (Stokes et. al, 2002, and Johnston et. al, 2000). However, even in moderately cracked elements, the penetration depth for a sufficient dose of lithium within the cement pore solution was only 50 mm (Stokes et. al, 2002, Johnston et. al, 2000, and Folliard et. al, 2012).

Topical applications of surface vapor barriers have proven an effective means of reducing ASR expansion in some concrete elements. Topical application of silane or siloxane can reduce internal relative humidity within concrete elements for 5 or more years (Berube et. al, 2002b). Additional researchers included elastomeric paint crack bridging surface treatments to control internal humidity in concrete with wide cracks (Drimalas et. al, 2012, Thomas et. al, 2012, and Fournier et. al, 2004).

2.7.1 Stark et. al (1993)

The mitigation of ASR in concrete pavements is mentioned by Stark (1993), in a study of concrete bridge decks and pavements. The monitoring program consisted of falling weight deflectometer (FWD) and internal relative humidity measurements (Stark et. al, 1993). Internal relative humidity was monitored through a method developed by Stark (1990) in which powder concrete samples were removed by drilling into the concrete and collecting samples at selected depths within the concrete (Stark et. al, 1993). The samples were then stored in a bottle and the equilibrium relative humidity within the bottle was measured with a probe (Stark et. al, 1993). Samples from various depths were then assembles to produce a relative humidity gradient with respect to depth of concrete (Stark et. al, 1993). Deflection measurements were taken before treatment and then again one year after treatment (Stark et. al, 1993). These measurements were then correlated to elastic modulus of the concrete, and used to monitor the progression of ASR within the concrete (Stark et. al, 1993). The surface treatments evaluated in the study included lithium, silane, and linseed oil. Unfortunately, only one year of monitoring was provided, and no conclusions were made on the effectiveness of the sealers (Stark et. al, 1993). However, Stark (1993) did conclude that FWD was a valid method of monitoring the deterioration of pavements due to ASR.

Stark (1993) reported that silane treatment of concrete pavements only provided a reduction in internal relative humidity within the top 0.5 to 1 inch of pavement. Research postulated that the silane was ineffective at mitigating expansion in concrete pavements due to moisture moving into the pavement from the subgrade. However, only one year of monitoring was available to develop this conclusion, which is not long enough to provide conclusive results on the efficacy of a surface treatment. The report also determined that topical lithium produced the

greatest reduction in expansion, again from one year of monitoring. Several publications have reported that the penetration of lithium into hardened concrete was not sufficient to provide a beneficial reduction in expansion (Stokes et. al, 2002, Johnston et. al, 2000, and Folliard et. al, 2012). More recent publications on the efficacy of topical silane mitigation in concrete transportation structures agree that silane provides a reduction in internal relative humidity (Berube et. al, 2002a, Drimalas et. al, 2012, and Thomas et. al, 2012). However, none of these publications specifically address concrete pavements treated with silane.

2.7.2   Berube (2002a)

Berube (2002a) provided conclusive results on the efficacy of silane and siloxane sealers from over 10 years of expansion and internal relative humidity monitoring of median barriers. The monitoring program involved selecting sections for treatment and control and then instrumenting the sections with expansion monitoring grids and internal relative humidity and temperature probes (Berube et. al, 2002a). Gage reference studs were affixed to the wall, with drilled points in the gage reference studs which were matched up with the points on the ends of a detachable mechanical strain (DEMEC) gage. The length-change between two gage reference studs was then used to monitor expansion. Gage reference studs were positioned for vertical and thickness length-change measurements. In addition holes were drilled in each section to monitor internal relative humidity and temperature with a commercial humidity probe (Berube et. al, 2002a).

Results from 10 years of monitoring indicate that both silane and siloxane produce a reduction in expansion and internal relative humidity for treated sections as compared to the control (Berube et. al, 2002a). The silane treatments were more effective than the siloxanes at reducing expansion. The treatments produced decreases in internal relative humidity for 6 years,

and then had reduced effectiveness (Berube et. al, 2002a). Therefore, Berube (2002a) recommends a reapplication of silane after 5 to 6 years of service. In addition, silane was effective in reducing expansion in moderately damaged median barriers for 10 years and 6 years, respectively. However, the siloxane treatment was less effective when used on severely damages sections, and only provided 1 to 2 years of protection (Berube et. al, 2002a). These results are based on the evaluation of concrete median barriers; however, they are applicable to concrete members with similar thickness and exposure conditions (Berube et. al, 2002a).

2.7.3   Berube (2002b)

Freezing and thawing cycles exacerbate the deterioration of damage in concrete which has cracked due to ASR (Berube et. al, 2002a). However, treating the samples with silane, siloxane, or linseed oil can protect the concrete from moisture, and therefore expansion (Berube et. al, 2002a). An extensive laboratory evaluation of the effectiveness of sealers on concrete samples affected with ASR and subjected to freezing and thawing cycles was conducted by Berube (2002a). Samples treated with silane, siloxane, or linseed oil and subjected to freezing and thawing and ASR expansion in the laboratory exhibited a reduction in expansion as compared to untreated control samples (Berube et. al, 2002a). A strong correlation between ASR expansion and internal relative humidity was also noted (Berube et. al, 2002b). Silane showed the greatest ability to reduce expansion in concrete which had ASR and was subjected to freezing and thawing cycles (Berube et. al, 2002b). The concrete sealed with linseed oil exhibited a reduction in expansion; however, the expansion still resulted in cracking when subjected to freezing and thawing (Berube et. al, 2002b). The results showed that any reduction in expansion correlated to a reduction in moisture, and therefore humidity, within the concrete after it was sealed (Berube et. al, 2002b).

2.7.4 Drimalas (2012)

Several mitigation methods were evaluated under the FHWA Alkali-Silica Reactivity (ASR) Development and Deployment Program. Preliminary results from this program are summarized by Drimalas (2012). The first mitigation evaluation involved several bridge columns in Texas which had expansion due to ASR, and the second involved a median barrier in Massachusetts which also exhibited ASR (Drimalas et. al, 2012). The column treatments included silane, or lithium applied through vacuum impregnation or electrochemical migration. The median barrier treatments included lithium, silane, penetrating membrane, and lithium vacuum impregnation (Drimalas et. al, 2012). The columns treated with lithium did not develop sufficient penetration of lithium or a reduction in expansion. Silane produced the only reduction in expansion for both the columns and median barrier (Drimalas et. al, 2012). As with all ASR field research, several years of monitoring are required to produce conclusive results. Both the columns and median barriers were monitored for 5 years; therefore, results on the sealer durability were not available. However, the results demonstrate the efficacy of silane in protecting concrete from moisture and reducing expansion.

2.7.5 Thomas (2012a)

Some additional research projects included under the FHWA Alkali-Silica Reactivity (ASR) Development and Deployment Program were reported by Thomas (2012a). These projects included a bridge structure in Maine, and a bridge in Vermont. Surface treatments evaluated in the study included silane or elastomeric paint (Thomas et. al, 2012). Several columns were also treated with lithium nitrate through either vacuum or electrochemical impregnation, or with topical silane. The bridge in Maine was treated in 2009 and the bridge in Vermont was treated in 2010.

Unfortunately, the study did not provide any preliminary results on the efficacy of the surface treatments.

These few case studies on the efficacy of surface treatment methods show promising results with methods such as silane, siloxane, and elastomeric paint. However, no conclusive case studies were available on the efficacy of surface treatments applied to pavement structures. There is concern that moisture will enter the concrete pavement from the subgrade. Especially after treatment, when a humidity gradient is present within the concrete pavement. The humidity gradient may provide suction, and draw moisture out of the subgrade. In addition, the pavement is subject to traffic wear which may reduce the effective life of the treatment. Unfortunately, at this time, no conclusive long term results were published on the efficacy of surface treatments applied to pavements affected by ASR.

CHAPTER 3: EXPERIMENTAL PROCEDURES

3.1 Research Plan

The goal of this research project is to catalog the reactivity of possible aggregate sources. Several cases of ASR have been identified in Arkansas since 2008. Most notable are the runway and taxiways at XNA regional airport, deteriorating concrete barriers along I49, pavement deterioration on I49, and joint failures on I30 Little Rock. In an attempt to prevent deleterious expansion due to alkali-silica reactivity ARDOT has funded this research project to test coarse and fine aggregate sources recognized in the Qualified Product List (QPL) by the Materials Division. To achieve this goal, material was gathered from 18 coarse aggregate sources and three fine aggregate sources. The reactivity of these materials were tested through traditional laboratory methods, AMBT (ASTM C1260) and CPT (ASTM C1293), and exposure block testing. Using these methods, potential reactive aggregates were identified.

3.2 Materials

Prior to the start of the project, the research team met with representatives from the Materials Division at ARDOT to determine which fine and coarse aggregates were to be included in the study. Eighteen coarse aggregates and three fine aggregates were chosen. Each aggregate is discussed in greater detail in the following sections.

3.2.1 Coarse Aggregates

Eighteen coarse aggregates were used in the testing process, four limestones, seven sandstones, four gravels, and three dolomites. All of the coarse aggregates are used in ARDOT construction projects. These aggregates are presented with their locations and ARDOT district, abbreviation, and physical properties in Table 3.2.1-1. The physical properties include specific gravity (S.G.) and absorption capacity (A.C.).

Table 3.2.1-1: Coarse Aggregate Locations and Physical Properties

Location	Abbreviation	Agg. Type	S.G.	A.C.	District
APAC Central – Hindsville Quarry	APHQ	Limestone	2.68	1.65	4
APAC Central – Sharp Quarry	APSQ	Limestone	2.68	0.40	9
APAC Central – Gravette	APG	Limestone	2.68	0.40	9
Midwest Lime Co.	MLC	Limestone	2.50	0.68	5
APAC Central – Jenny Lind, AR	APJL	Sandstone	2.68	1.65	4
Rogers Group, Greenbrier AR	RGG	Sandstone	2.20	0.40	8
Hamilton Aggregates Inc. – Center Ridge, AR	HMCR	Sandstone	2.20	0.47	8
Rogers Group, Cabot AR	RGC	Sandstone	2.36	1.76	6
Hanson Aggregates – Wilton, AR	HAGG	Sandstone	2.80	1.23	3
Hamilton Agg, Pryor Mt. Quarry	HMPM	Sandstone	2.69	1.12	5
Martin Marietta Materials	MMM	Sandstone	2.69	1.00	6
Razor Rock Materials – Harrisburg Pit Plant 671	RRM	Gravel	2.25	0.60	10
Wooten Sand & Gravel – Fouke, AR	WSG	Gravel	2.66	1.39	3
APAC Tennessee– White Oak Pit	APT	Gravel	2.25	0.60	1
Standard Gravel Co.	SGC	Gravel	2.53	1.43	7
Vulcan Materials Company - Judsonia, AR	VMJ	Dolomite	2.69	1.38	5
Arkansas Quality Stone	AQS	Dolomite	2.75	1.01	5
Vulcan Materials Company - Black Rock, AR	VMBR	Dolomite	2.60	1.20	10

3.2.2 Fine Aggregates

Three fine aggregates were used in the testing process, one fine aggregate was an excavated gravel fine and the remaining two were Arkansas River Sand. All of the fine aggregates are used

in ARDOT construction projects. These aggregates are presented with their locations and ARDOT district, abbreviation, and physical properties in Table 3.2.2-1. The physical properties include specific gravity (S.G.) and absorption capacity (A.C.).

Table 3.2.2-1: Fine Aggregate Locations and Physical Properties

Location	Abbreviation	Agg. Type	S.G.	A.C.	District
APAC Central – Pools 12 -14	AP12	River sand	2.63	0.48	4
Standard Gravel Co.	SGC	Gravel	2.60	0.30	7
Jeffery Sand Co.	JSC	River sand	2.61	0.48	8

3.2.3 Portland Cement

For the AMBT a standard Type I/II portland cement was used for casting mortar bars. The cement had an alkali content less than 0.60% and was passed through an 850 μm (No. 20) sieve to remove large agglomerations of cement, as specified in ASTM C1260. For the CPTs and exposure blocks, a high alkali Type I/II portland cement was used for casting. This cement had an elevated alkali content of 0.89% in accordance with ASTM C1293 specification ($0.9 \pm 0.1\%$). Furthermore, NaOH pellets were added to the mixing water to increase the alkali content of the mixture to 1.25% Na_2O_{eq} by mass of cement (ASTM C1293). Table 3.2.3-1 presents the chemical composition of the high alkali cement used during testing.

Table 3.2.3-1: High Alkali Cement Properties

Chemical Compounds	Percent
SiO_2 (%)	19.70
Al_2O_3 (%)	4.10
Fe_2O_3 (%)	2.50
CaO (%)	61.90
MgO (%)	3.60
SO_3 (%)	3.50
Equivalent Alkalis (%)	0.89
C_3S (%)	56.00
C_3A (%)	7.00

3.3 Testing Procedure

Laboratory tests were conducted to evaluate the potential alkali-silica reactivity of the aggregates used in Arkansas. The laboratory tests followed existing ASTM standards for proposed test methods from the literature. The tests were used to evaluate the potential reactivity of aggregates.

3.3.1 Accelerated Mortar Bar Test (AMBT)

The AMBT has a short test duration (16 days) which allows for rapid evaluation of aggregates for potential alkali-silica reactivity. The test method was conducted in accordance with ASTM C1260 Standard Test Method for Potential Alkali Reactivity of Aggregates (Mortar-Bar Method). The aggregate was first sieved to the standard gradation and then a standard batch of mortar was mixed. When evaluating coarse aggregates, the aggregate was first crushed and then sieved to match the gradation. The aggregate gradation and batch weights to produce three mortar bars are summarized below in Table 3.3.1-1.

Table 3.3.1-1 ASTM C1260 Standard Gradation and Batch Weights.

	No. 8	No. 16	No. 30	No. 50	No. 100	Cement	Water
Fraction	10	25	25	25	15	--	0.47
Mass (g)	99	247.5	247.5	247.5	148.5	440	206.8

The standard batch produced enough mortar for three mortar-bars with standard dimensions of 25 x 25 x 285 mm (1 x 1 x 11.25 in). The mortar bars were cured for 24 hours in an environmental chamber with ambient temperature of $23 \pm 2°$ C and 95% RH. The mortar-bar molds are displayed in Fig. 3.3.1-1 (left), and typical cured mortar-bars are displayed in Fig. 3.3.1-1 (right).

Fig. 3.3.1-1 Mortar-Bar mold with gage studs (left) and mortar-bar mold with three mortar-bars (right).

The mortar bars were removed from the molds after 24 hours of curing. The mortar-bars were then placed in 80° C water for 24 hours. A typical set of cured mortar bars are displayed in Fig. 3.3.1-2 (left), and a typical set of mortar-bars in the storage containers are displayed in Fig. 3.3.1-2 (right).

Fig. 3.3.1-2 Mortar-bars (left) and mortar-bar in storage container (right).

The initial length-change of each mortar-bar was measured after 24 hours in the water bath. The mortar-bar temperature equilibrates when exposed to air, and the length-change for each mortar-bar is measured within 15 ± 5 seconds of being removed from the container to reduce thermal shrinkage. The three mortar-bars were measured, and then moved to a container of 1N sodium hydroxide (NaOH) solution which was stored at 80° C. The mortar-bars were stored in the sodium hydroxide (NaOH) solution for 14 days. Length-change measurements were ascertained a minimum of three times during the 14-day storage period.

The sodium hydroxide (NaOH) solution was produced by dissolving 40 grams of sodium hydroxide (NaOH) pellets in 900 ml of water. After the sodium hydroxide (NaOH) pellets dissolved, the solution was diluted to obtain 1.0 L of solution. The required volume of solution within the storage container is four times the volume of the mortar-bars. The sealed containers were stored in a water bath at 80° C for the duration of the test.

The final length-change was determined after 14 days of storage. The interim measurements were ascertained at 4 and 7 days of storage. The length-change was measured with a comparator and digital gage with a precision of 0.001% of the effective gage length. The length-change of the three mortar-bars was averaged, and reported to the nearest 0.01%. The final length-change was then compared to the expansion limits. Expansions less than 0.10% at 16 days indicate

innocuous aggregates. Expansion between 0.1 and 0.2% at 16 days require additional information to establish aggregate reactivity. In addition, the specification allows a test duration of 28 days for samples with 14 day expansions between 0.1% and 0.2%. Mortar-bars with 14 or 28-day expansion greater than 0.20% indicate potentially deleteriously expansive aggregates.

The results of each AMBT was compiled and plotted with time (days) on the abscissa and length-change (percent) on the ordinate axis. The AMBT storage conditions necessary to accelerate ASR and prevent alkali-leaching are harsh and often produce excessive expansion in aggregates with proven field performance. Therefore, the AMBT results were compared to CPT results to confirm the aggregate classification.

The AMBT method accelerates the development of ASR by storing the mortar-bars at 80° C. The sodium hydroxide solution prevents alkalis from leaching from the mortar bars during storage. The storage environment accelerates the development of ASR and provides results in 16 days. The test is effective when evaluating preventative measures against ASR and can produce conservative estimates on the required level of prevention. The AMBT method allows for the evaluation of cement-aggregate mixtures with a partial replacement of cement with SCMs. The test was conducted in accordance with ASTM C1567 Standard Test Method for Determining the Potential Alkali-Silica Reactivity of Combinations of Cementitious Materials and Aggregate (Accelerated Mortar-Bar Method). The test method is identical to the AMBT with the exception of the cementitious materials. Fly ash, silica fume, slag cement, and metakaolin are evaluated at various replacement rates to determine the minimum safe replacement rate which will prevent ASR expansion. A 16-day expansion less than 0.10% signifies a safe replacement rate. Some SCMs, such as fly ash, delay alkali-silica reactivity and the AMBT duration is extended to 28 days to allow for any delayed reactions.

Fine aggregate from the Arkansas River was evaluated in combination with Class C fly ash. The fly ash was sourced from the same location as the fly ash used within the median barrier, and had a similar chemical composition. Tests were conducted at several fly ash replacement rates ranging from 10% to 40% replacement by weight of cement. The safe replacement rate was determined through an evaluation of replacement rates below and above the safe range. The replacement rate which provided 30-day expansion below 0.10% was then determined.

3.3.2 Concrete Prism Test (CPT)

The CPT was conducted in accordance with ASTM C1293. The requirements for a CPT standard concrete mixture were provided in the specification. The coarse aggregate was sieved to the standard gradation as specified in ASTM C1293. The concrete mixture required a coarse aggregate oven-dry-rodded unit volume of 0.70% $\pm$ 0.2%. The water to cementitious material ratio (w/cm) was specified between 0.42 and 0.45 (by mass). The volume fraction of sand was selected to produce a unit volume of concrete. The cement content was specified as 420 kg/m^3 (708 lb/yd^3), and the cement alkali content was limited to 0.90% $\pm$ 0.1% Na_2O_e. The alkali content was increased to 1.25% Na_2O_e by the addition of sodium hydroxide (NaOH) to the mixture water before batching. The required sodium hydroxide was determined from the cement and cement alkali content. The mixture proportions used for the CPT are summarized below in Table 3.3.2-1.

After the concrete mixture was batched, slump, unit weight, and air content tests were performed in accordance with ASTM C143, and ASTM C138, respectively. Concrete was then placed into the prism molds in two lifts, with rodding and tamping after each lift to ensure sufficient compaction. The prisms are 75 x 75 x 285 mm (3 x 3 x 11.25 in) and are displayed below in 3.3.2-

1. A total of three prisms were required for each concrete mixture evaluated. After troweling the

top of the prisms, the molds were placed and cured in the environmental chamber for 24 hours.

After curing, the prisms were removed from the molds and the initial length-change was

measured using a length-change comparator with a digital gage. The prisms were then placed in

the storage containers and placed in a water bath at 38° C. The duration of length-change

monitoring was one year, with interim readings taken at 7, 28, 56 days, 3, 6, 9, and 12 months.

The storage containers were removed from the water bath 16 ± 4 hours before length-change

readings to allow the temperature of the prisms to equilibrate to room temperature. The storage

containers must also maintain a high RH (ASTM, 2012). The containers used for storage had

dimensions of 225 x 300 x 112 mm (9 x 12 x 4.5 in.) with a water tight cover. The prisms were

elevated 25 mm (1 inch) from the bottom of the container with small blocks of wood, and then

12.5 mm (1/2 inch) of water was placed in the bottom of the container. The containers were sealed

and placed in a water bath to maintain a temperature of 38° C.

Fig. 3.3.2-1 Typical CPT prism molds (left), and storage containers (right).

Table 3.3.2-1 ASTM C1293 mixture design specifications and typical batch weights.

	Notes	kg/m^3 (lb/yd^3)
Coarse	0.70 ± 0.05 D.R.U.W.	1110 (1871)
1/2	33%	370 (624)
3/8	33%	370 (624)
No. 4	33%	370 (624)
Sand	F.M ~ 2.7	629 (1060)
Cement	~ 0.90% Na$_2$O$_e$	420 (708)
Water	w/cm ~ 0.45	189 (319)
NaOH	1.25% Na$_2$O$_e$	1.90 (3.20)

After one year of monitoring, results from the three prisms were averaged to produce a single plot. The data were plotted with time (days) on the abscissa and length-change (percent) on the ordinate axis. A one-year expansion greater than 0.04% indicates a concrete mixture with potentially deleterious expansion. The CPT provides the most reliable accelerated method of evaluating aggregates for alkali-silica reactivity.

3.3.3 Exposure Site

In addition to laboratory testing methods, ASTM C1260 and ASTM C1293, an outdoor exposure site was utilized to simulate field performance of concrete as seen in Figure 3.3.3-1. The site is located on the southern side of the Engineering Research Center at the University of Arkansas in Fayetteville, Arkansas. There are 21 concrete exposure blocks made with aggregate from quarries around the state of Arkansas. There is coarse aggregate from eighteen different sources and fine aggregate from three different sources. These blocks nominally measure 710 mm

long by 380 mm high and 380 mm wide (28 in x 15 in x 15 in) and rest on a limestone base to avoid direct contact with the ground.

Figure 3.3.3-1: Outdoor Exposure Site (Fayetteville, Arkansas)

Concrete mix proportions are modeled after the CPT mixtures design prescribed in ASTM C1293. This allows the relationship between laboratory conditions and actual behavior in the field to be directly examined. Also, it mimics the casting procedure used by Folliard et al. (2006) and can be used for comparison. The fresh concrete was mixed in a large rotating drum mixer then placed, rodded, and compacted into the mold in three equal lifts. The molds were made of wood panels fastened on each end by a set of stainless steel threaded rods secured by washers and bolts. After casting, two stainless steel gauge studs were positioned in 23 mm (15/16 inch) diameter holes on all lateral and longitudinal faces of the block. Gauge studs were dimensioned with an 8 mm (5/16 inch) diameter, 75 mm (3 inch) long, and 3 mm deep drilled hole, where the point of the Detachable Mechanical Strain (DEMEC) gage was inserted. These studs were horizontally

orientated at 20 inches on center on the longitudinal faces and vertically orientated 8 inches on center on the lateral faces, as shown in Figure 3.3.3-2 and Figure 3.3.3-3. After removing the mold, the block is moved to the exposure site where the zero reading and temperature are recorded. Subsequent readings were recorded once a month and were usually recorded in the early morning when there was the least amount of thermal expansion (direct sunlight). The readings were taken when there was no precipitation.
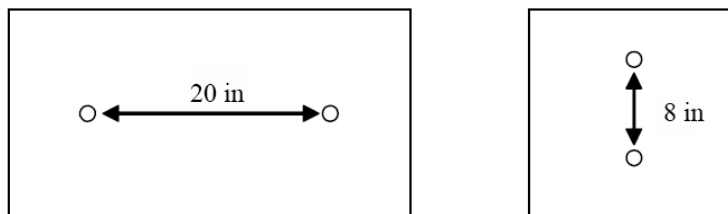

Fig 3.3.3-2: Longitudinal (left) and lateral (right) Gauge Stud placement diagram

Figure 3.3.3-3: Outdoor Exposure Block (Fayetteville, Arkansas)

CHAPTER 4: RESULTS

4.1 Introduction

This chapter presents the results from the AMBT, CPT and exposure blocks. This chapter compares the results of the AMBT and CPT test to the results from the exposure blocks. Using the results from the three tests, the research team will identify the potentially deleterious aggregates used in the state of Arkansas.

4.2 AMBT

The accelerated mortar-bar test was conducted in accordance with ASTM C1260. Mortar mixtures were designed after mix proportions specified in ASTM 1260 and aggregates were crushed and sieved to match the grading requirement stated in the specifications. The mortar-bars were cured for 24 hours before initial readings then stored at 80 ± 2 ^{o}C (176 ± 3.6 ^{o}F). For each mixture tested, three 25 x 25 x 285 mm (1 x 1 x11.25 in) bars were cast. Mortar-bars were subject to a testing duration of 28 days, although the 14 day expansion determines, based on ASTM C1260, whether the aggregate is classified as inert, potentially reactive, or deleteriously reactive. When using SCMs (ASTM C1567) many research examine the 28 day expansion. Ongoing research at the University of Arkansas examined the effect of SCMs on mortar expansion, and therefore readings were taken out to 28 days of age for this project.

For all aggregates, the strain was averaged between the three mortar-bar specimens. Specimens with a percent strain below 0.1 percent at 14 days indicate an inert aggregate. Percent strains between 0.1 and 0.2 percent at 14 days reflect potentially reactive aggregates. If expansion exceeded 0.2 percent at 14 days, the aggregate was considered deleteriously reactive.

4.2.1 AMBT - Fine Aggregate

Three different fine aggregate sources were evaluated using the AMBT. The AMBT expansion curves for all fine aggregates sources are shown in Figure 4.2.1-1. The three fine aggregates were discussed in greater detail in Section 3.2.1. At 14 days of age, the three fine aggregates were classified as potentially reactive since the expansion was between 0.1 and 0.2 percent. There was little difference between the performances of the three fine aggregates. Based on these findings, further testing with the CPT would be recommended.
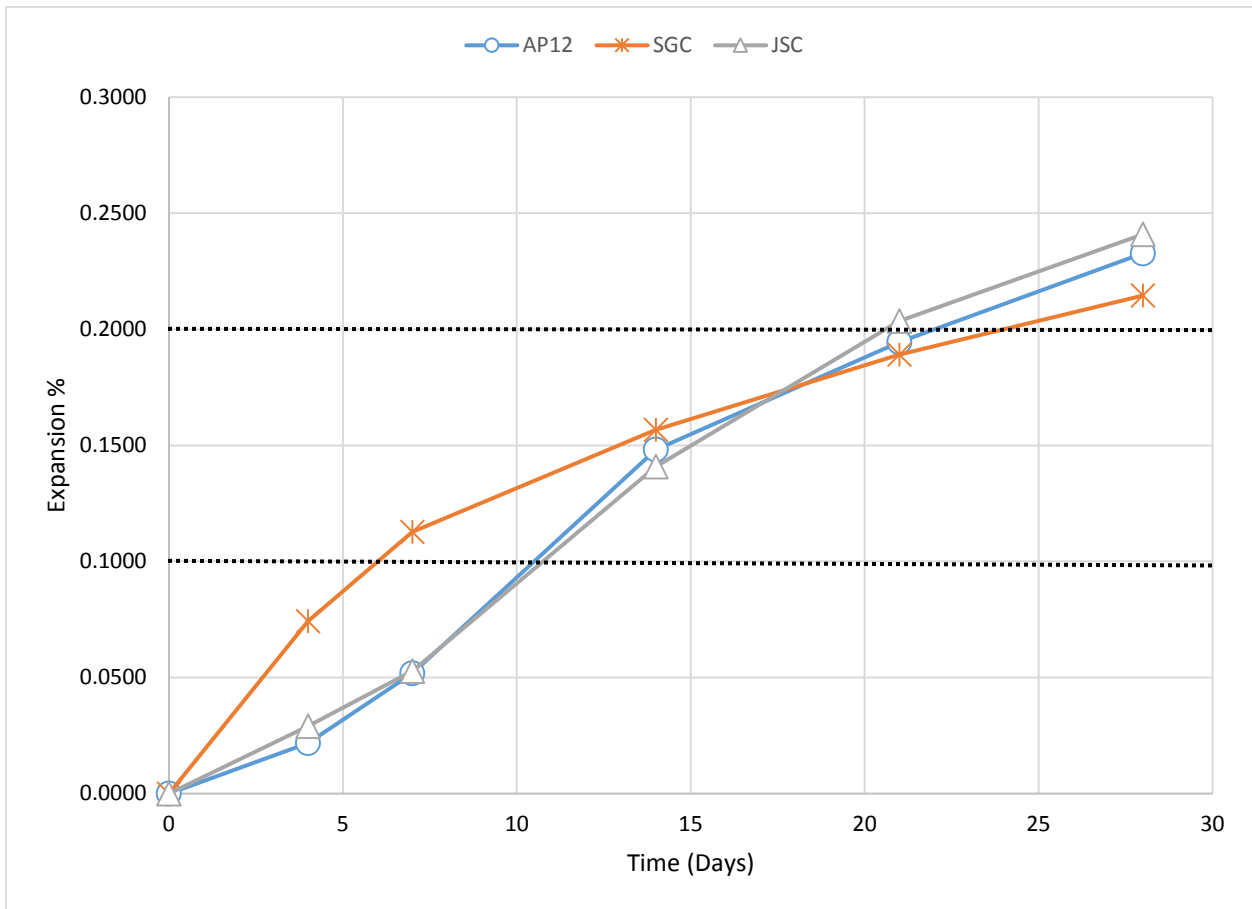

Figure 4.2.1-1: Average Strain (%) of AMBT Containing Fine Aggregate

4.2.2 AMBT - Sandstone

Seven different sandstone sources were evaluated using the AMBT. The AMBT expansion curves for all aggregates are shown in Figure 4.2.2-1. The seven sandstones were discussed in greater detail in Section 3.2.1.   The results show that there are no strains greater than or equal to 0.05 percent at 14 days. Based on the results, all sandstones would be classified as inert. Further testing using the CPT would not be necessary based on the results.
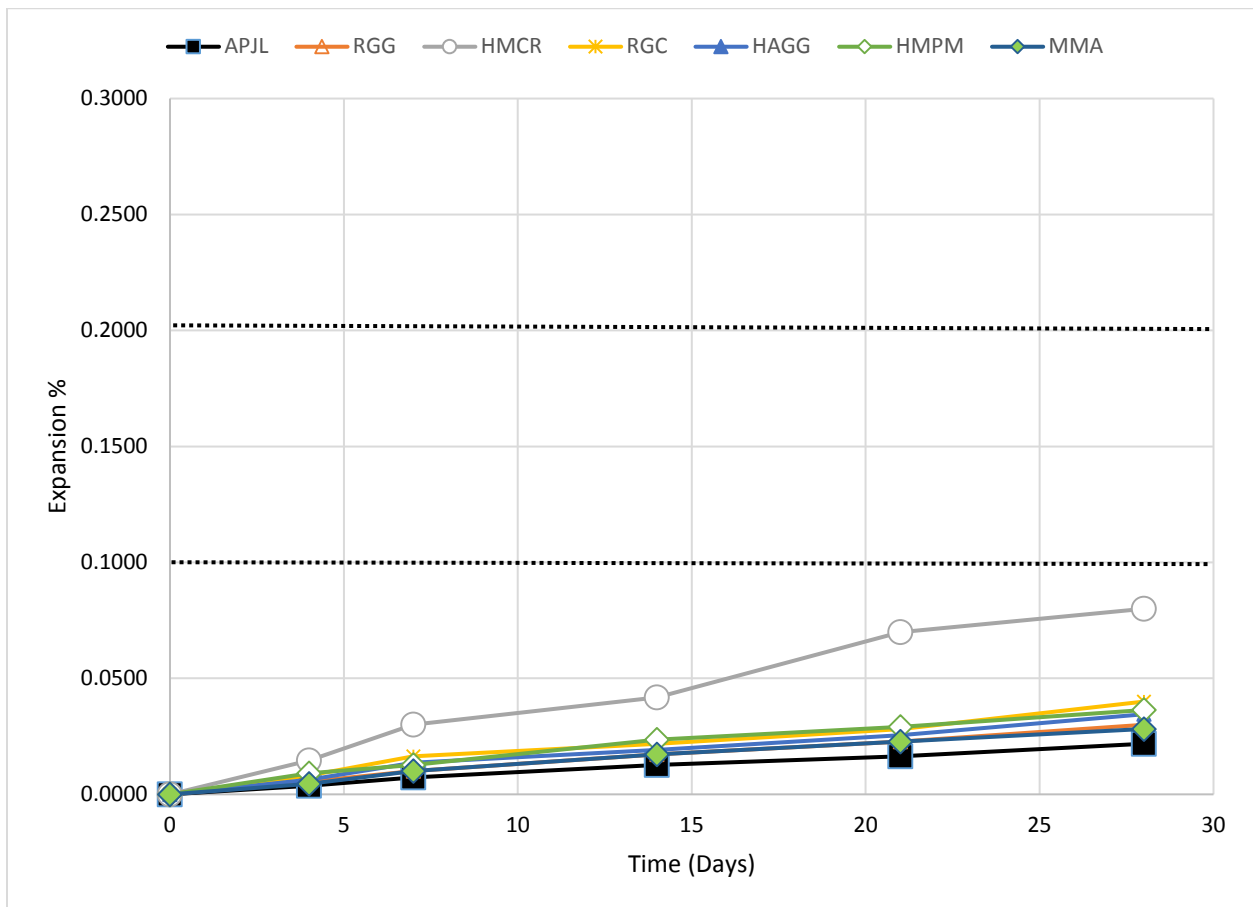

Figure 4.2.2-1: Average Strain (%) of AMBT Containing Sandstone Aggregate

4.2.3 AMBT - Limestone

Four different limestone sources were evaluated using the AMBT. The AMBT expansion curves for all limestone aggregates are shown in Figure 4.2.3-1. The four limestones were discussed in greater detail in Section 3.2.1. Based on the AMBT data, all limestone is classified as inert because the greatest 14 day was approximately 0.05 percent. Of the four specimens MLC appears to be the most reactive and the expansion reaches 0.1 percent at 28 days of age. However, CPT would not be necessary for the limestones since the expansion was less than 0.1 percent for all specimens at 14 days of age.
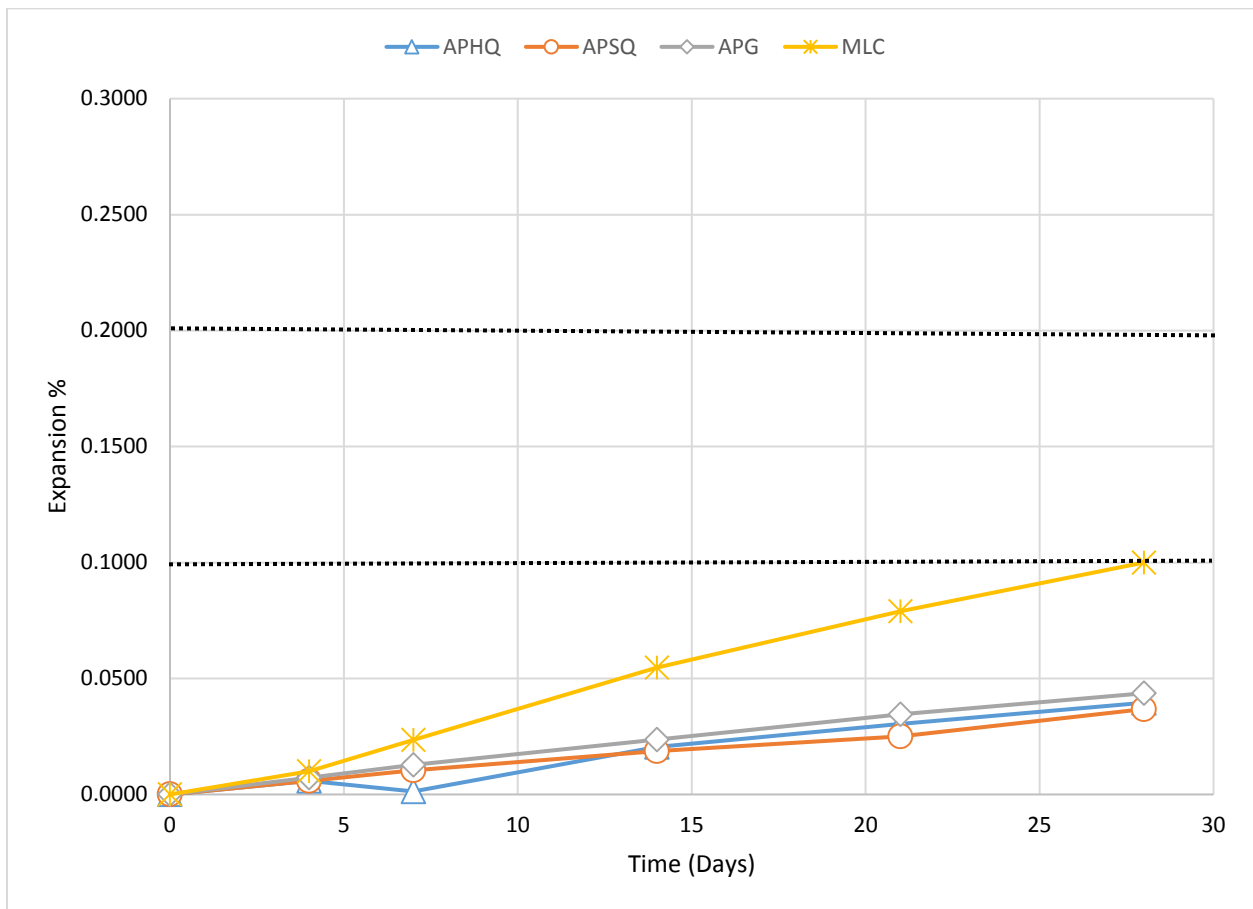

Figure 4.2.3-1: Average Strain (%) of AMBT Containing Limestone Aggregate

4.2.4 AMBT - Dolomite

Three different dolomite sources were evaluated using the AMBT. The AMBT expansion curves for all dolomite aggregates are shown in Figure 4.2.4-1. The three dolomites were discussed in greater detail in Section 3.2.1. Based on the AMBT data, all dolomite sources are classified as inert because the greatest 14 day was approximately 0.04 percent. Therefore, the CPT would not be necessary for the dolomite aggregates since the expansion was less than 0.1 percent for all specimens at 14 days of age.
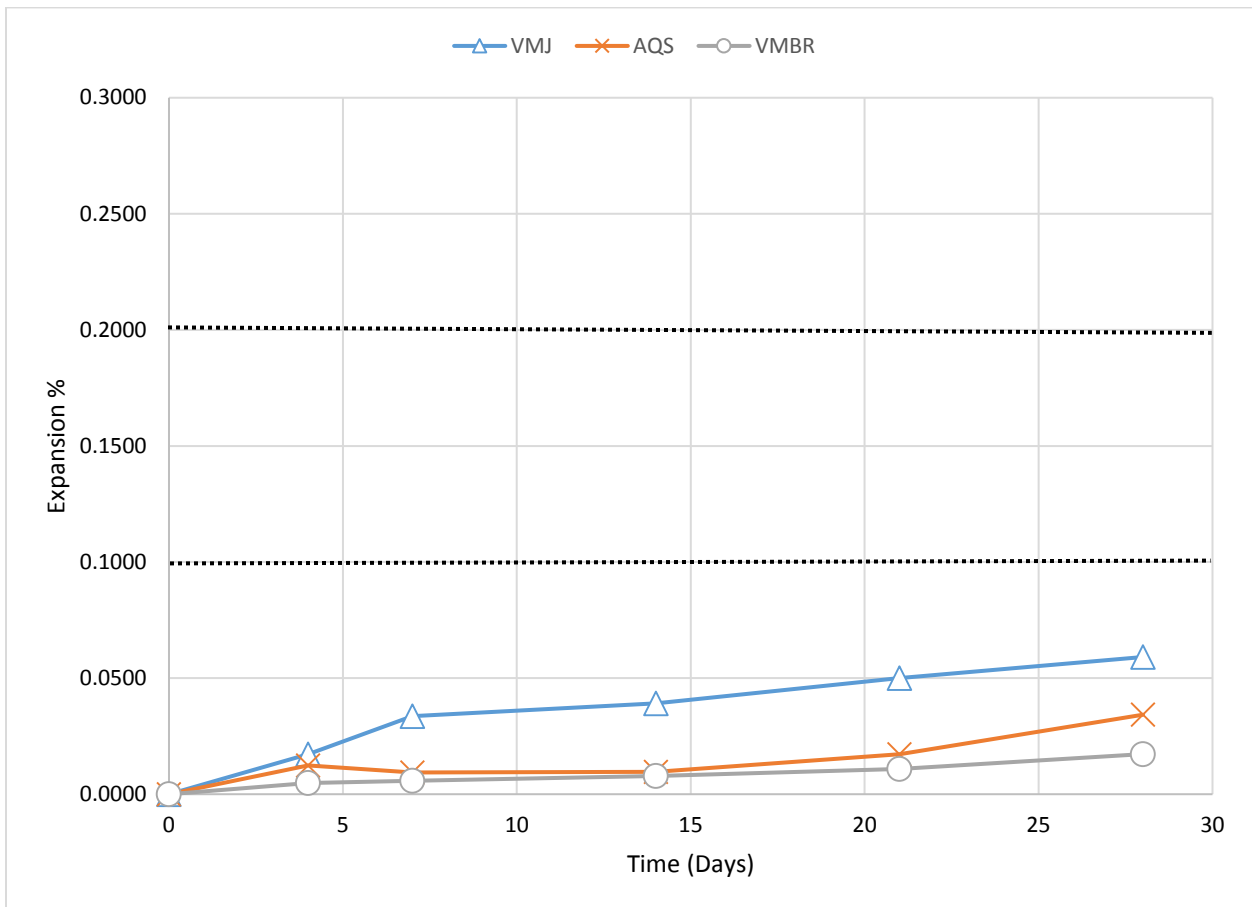

Figure 4.2.4-1: Average Strain (%) of AMBT Containing Dolomite Aggregate

4.2.5 AMBT - Gravel

Four different gravel sources were evaluated using the AMBT. The AMBT expansion curves for all gravel aggregates are shown in Figure 4.2.5-1. The four gravels were discussed in greater detail in Section 3.2.1. Based on the AMBT data, all gravel sources are classified as inert because the greatest 14 day was approximately 0.08 percent. Therefore, the CPT would not be necessary for the gravel aggregates since the expansion was less than 0.1 percent for all specimens at 14 days of age.
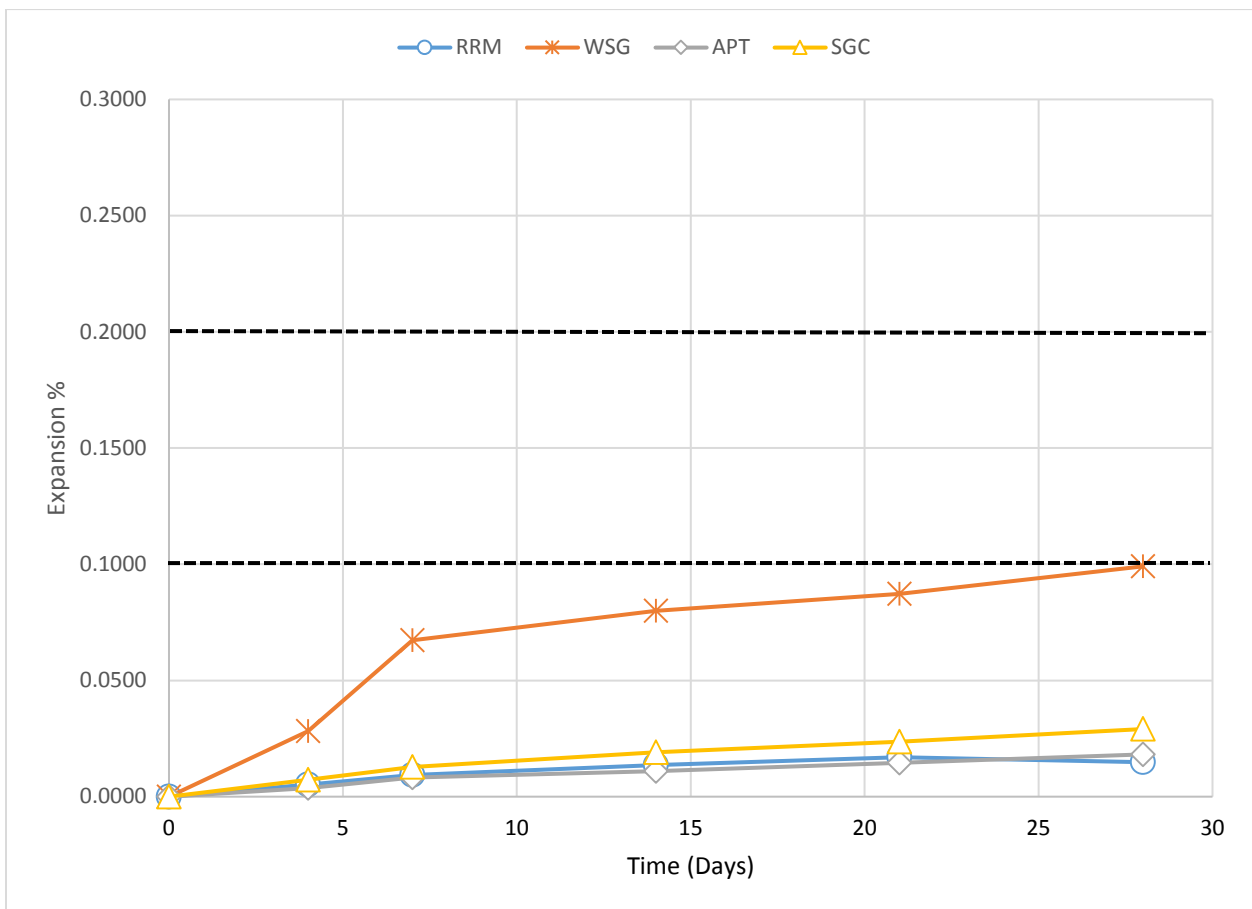

Figure 4.2.5-1: Average Strain (%) of AMBT Containing Gravel Aggregate

4.3 Concrete Prism Test (CPT)

The CPT was conducted in accordance with ASTM C1293. The mixture proportion were specified in ASTM 1293. The aggregate gradations also adhered to ASTM C1293 specifications. For each mixture tested, three 75 x 75 x 285 mm (3 x 3 x 11.25 in) prisms were cast. Concrete prisms were subject to a testing phase between one to two years. For some specimens, testing was stopped if expansion exceeded 0.04 percent. Expansions greater than 0.04 percent at 1 year are an indicator of a potentially deleterious reaction.

4.3.1 CPT - Fine Aggregate

Three different fine aggregate sources were evaluated using the CPT. The CPT expansion curves for all fine aggregates sources are shown in Figure 4.3.1-1. The three fine aggregates were discussed in greater detail in Section 3.2.1. At 360 days of age, the three fine aggregates were not deleteriously reactive since the expansion was less than 0.04 percent. However, previous research at the University of Arkansas has shown that Arkansas River Sand (AP12 and JSC) is reactive (Deschenes, Jr. and Hale, 2017a). These differences in results on the same fine aggregate sources highlight the limitations in the current tests.
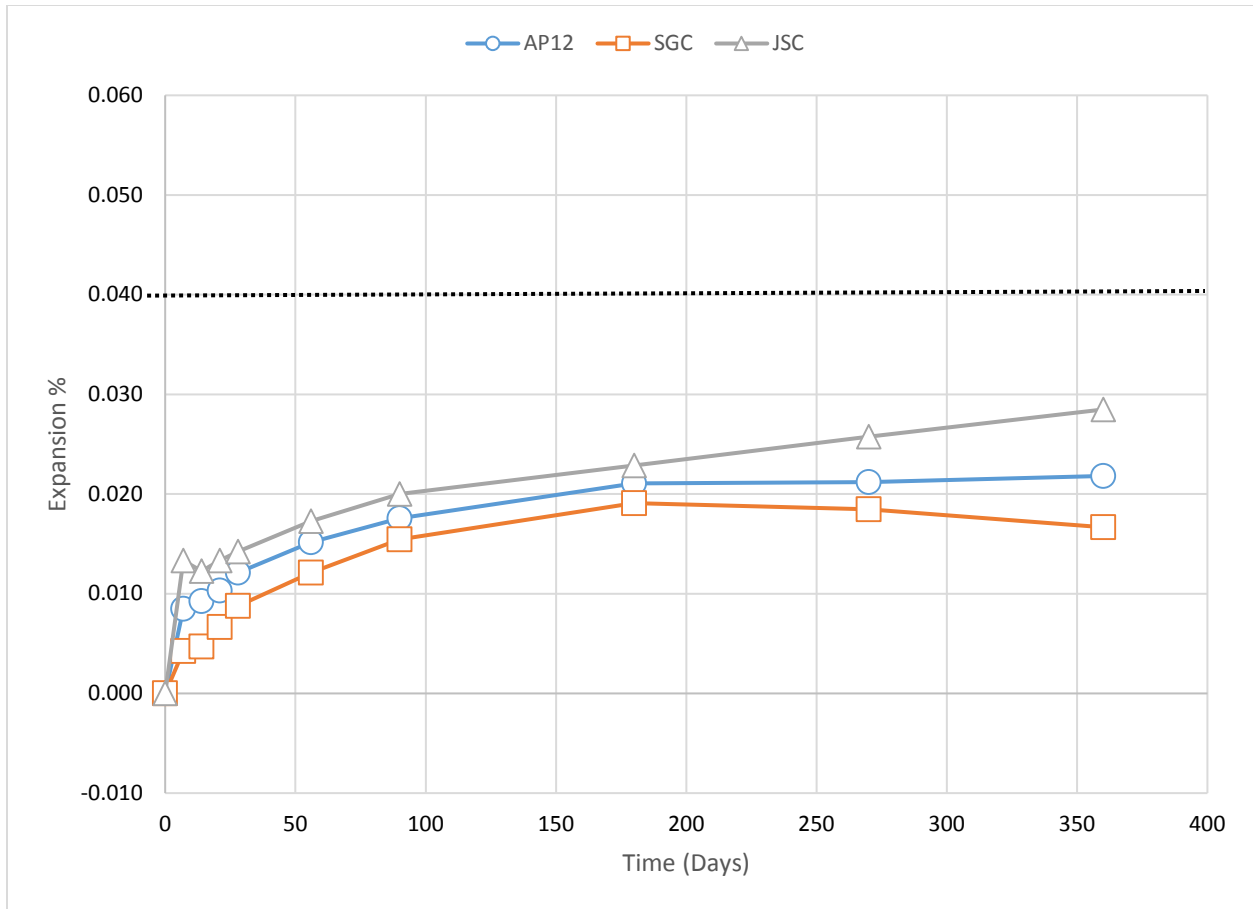
Figure 4.3.1-1: Average Strain (%) of CPT Containing Fine Aggregate

4.3.2 CPT - Sandstone

Seven different sandstone sources were evaluated using the CPT. The CPT expansion curves for all aggregates are shown in Figure 4.3.2-1. The seven sandstones were discussed in greater detail in Section 3.2.1. The results show that four of the seven sandstones are reactive because their strains greater than or equal to 0.04 percent at 360 days. Those four sandstones were HAGG (Hanson Aggregates), HMCR (Hamilton Aggregates, Center Ridge), APJL (APAC Central), and HMPM (Hamilton Aggregates, Pryor Mountain). The concrete containing the four

sandstones was classified as having potentially deleterious expansion. The remaining sandstones would be classified as inert.
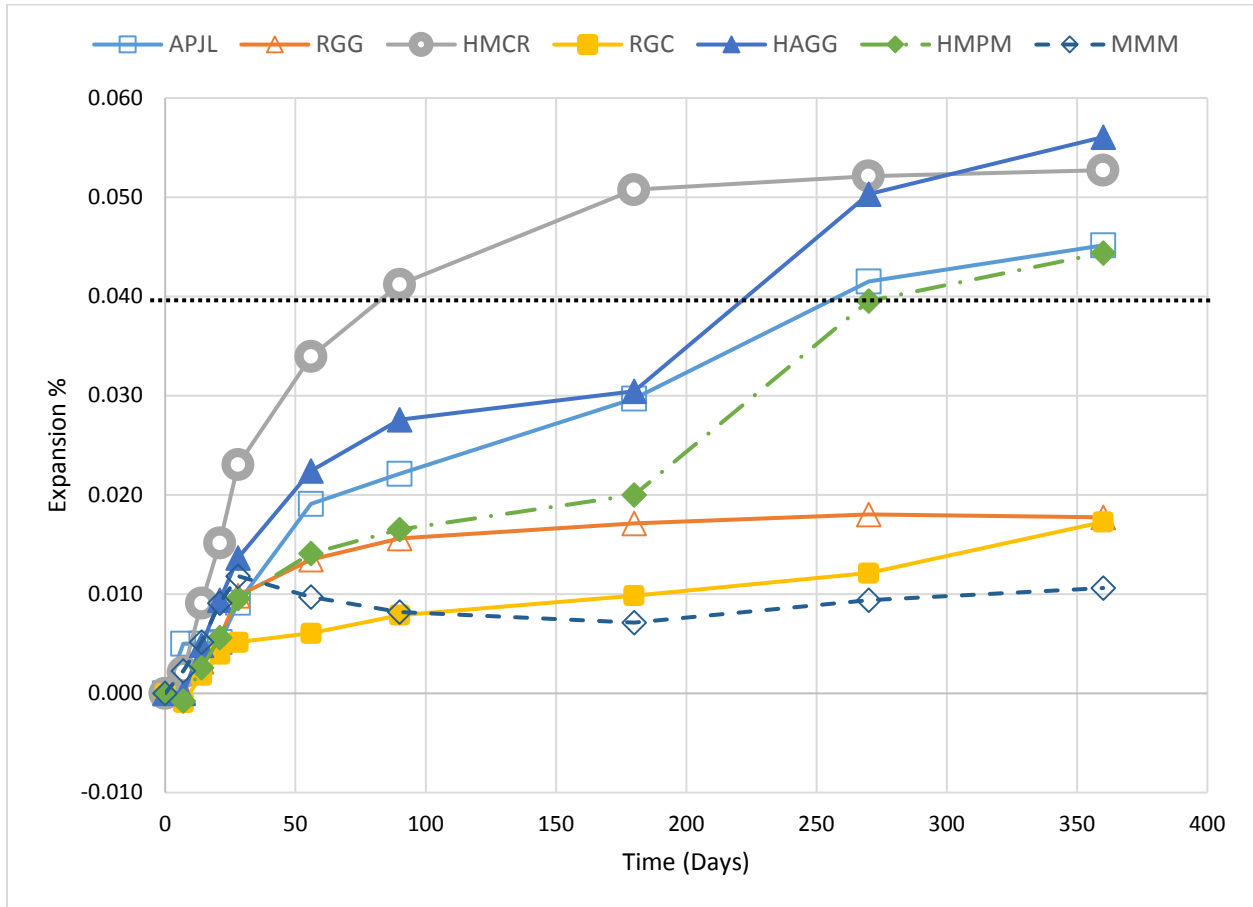

Figure 4.3.2-1: Average Strain (%) of CPT Containing Sandstone Aggregate

4.3.3 CPT - Limestone

Four different limestone sources were evaluated using the CPT. The CPT expansion curves for all aggregates are shown in Figure 4.3.3-1. The four limestones were discussed in greater detail in Section 3.2.1. The results show that one of the four limestones, MLC (Midwest Lime

Company), was classified as having potentially deleterious expansion because the strain was greater than 0.04 percent at 360 days. The remaining limestones would be classified as inert.
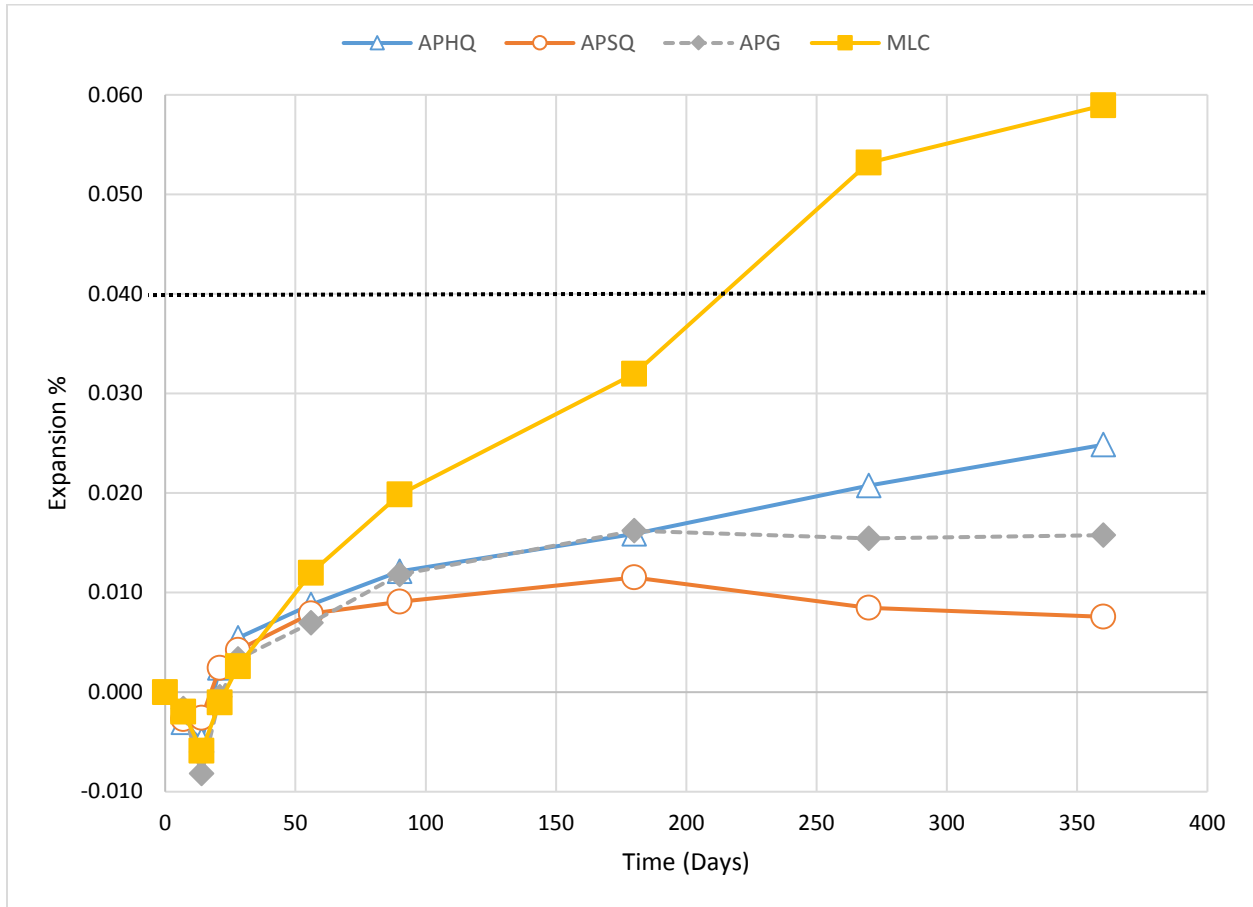
Figure 4.3.3-1: Average Strain (%) of CPT Containing Limestone Aggregate

4.3.4 CPT - Dolomite

Three different dolomite sources were evaluated using the CPT. The CPT expansion curves for all aggregates are shown in Figure 4.3.4-1. The three dolomites were discussed in greater detail in Section 3.2.1. The results show that all the dolomite aggregates would be classified as inert.

Figure 4.3.4-1 Average Strain (%) of CPT Containing Dolomite Aggregate

4.3.5 CPT - Gravel

Four different gravel sources were evaluated using the CPT. The CPT expansion curves for all aggregates are shown in Figure 4.3.5-1. The four gravels were discussed in greater detail in Section 3.2.1. The results show that all the gravel aggregates would be classified as inert. However, the spec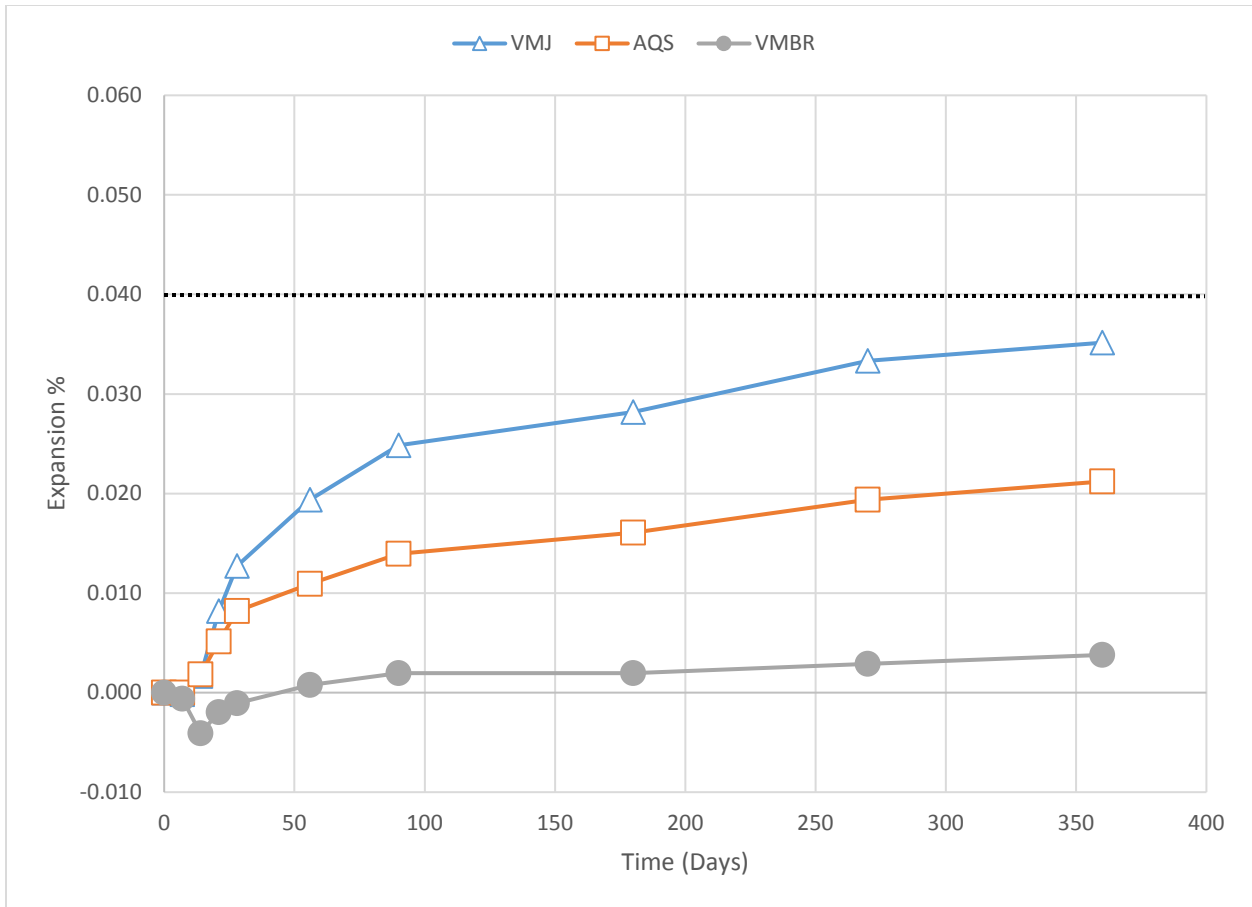imens cast with WSG had an expansion of 0.038 percent which is approximately twice the expansion of the other gravel specimens.
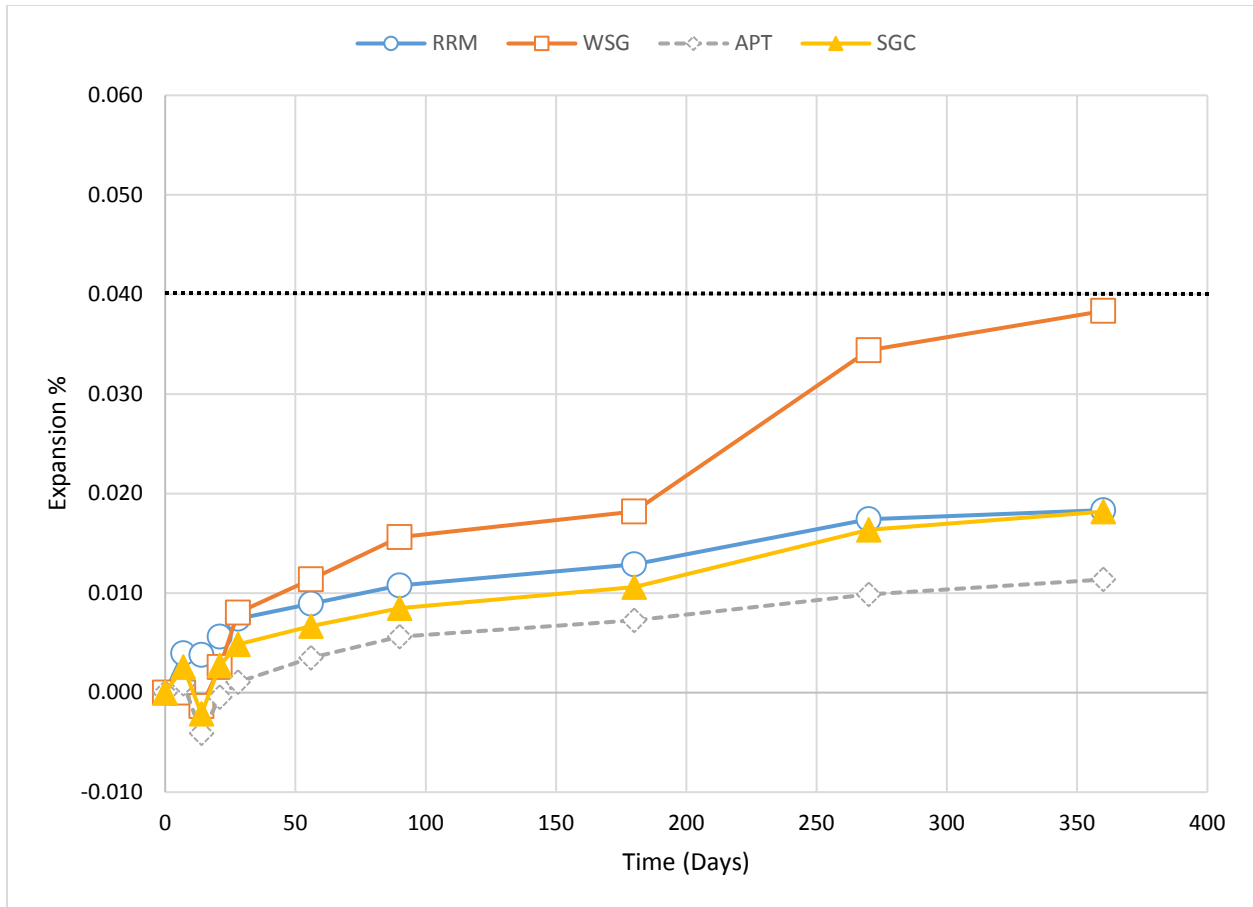
Figure 4.3.5-1: Average Strain (%) of CPT Containing Gravel Aggregate

4.4 Exposure Blocks

Each of the aggregates examined in the AMBT and CPT were also used to cast exposure blocks. The exposure blocks were proportioned using the CPT mixture design in accordance with ASTM 1293. The blocks nominally measured 710 mm long by 380 mm high and 380 mm wide (28 in x 15 in x 15 in) and were placed on a limestone base to avoid direct contact with the ground. The site was located on the southern side of the Engineering Research Center at the University of Arkansas in Fayetteville, Arkansas. The measurements reported were the averaged strain data from the longitudinal faces of the block (Figure 3.3.3-2). This face was chosen because this is the side of the block with the most surface area. The expansion data were corrected for temperature and

normalized for ambient temperature 21°C (70°F) as described in Section 3.3.3. Expansions exceeding 0.04% observed during the testing cycle are classified as potentially deleterious reaction.

4.4.1 Exposure Blocks - Fine Aggregate

The expansion data of the exposure blocks containing the three fine aggregates are shown in Figure 4.4.1-1. The three fine aggregates were discussed in greater detail in Section 3.2.1. The blocks are approaching two years of age and their strains are not greater than 0.04%. Based on this expansion, the three fine aggregates included in this study would be classified as not deleteriously reactive. However, the Standard Gravel Company (SGC) specimen has an expansion 0f 0.34% and exhibits a trend line that will exceed 0.4% by 700 days of age.

Figure 4.4.1-1: Average Strain (%) of Exposure Block Containing Fine Aggregate

4.4.2 Exposure Blocks - Sandstone

The expansion data of the exposure blocks containing the seven sandstone aggregates are shown in Figure 4.4.2-1. The seven sandstone aggregates were discussed in greater detail in Section 3.2.1. The blocks are approaching two years of age and their strains are not greater than 0.04%. Based on this expansion, the sandstone aggregates included in this study would be classified as not deleteriously reactive.

Figure 4.4.2-1: Average Strain (%) of Exposure Block Containing Sandstone Aggregate

4.4.3 Exposure Blocks - Limestone

The expansion data of the exposure blocks containing the four limestone aggregates are shown in Figure 4.4.3-1. The four limestone aggregates were discussed in greater detail in Section 3.2.1. The blocks are approaching two years of age and their strains are not greater than 0.04%. Based on this expansion, the limestone aggregates included in this study would be classified as not deleteriously reactive. However, it is important to note that the Midwest Line Company (MLC) specimen had the highest expansions in the AMBT, CPT, and exposure blocks. Their exposure block results show an expansive trend that will exceed 0.04 percent before 2 years of age.
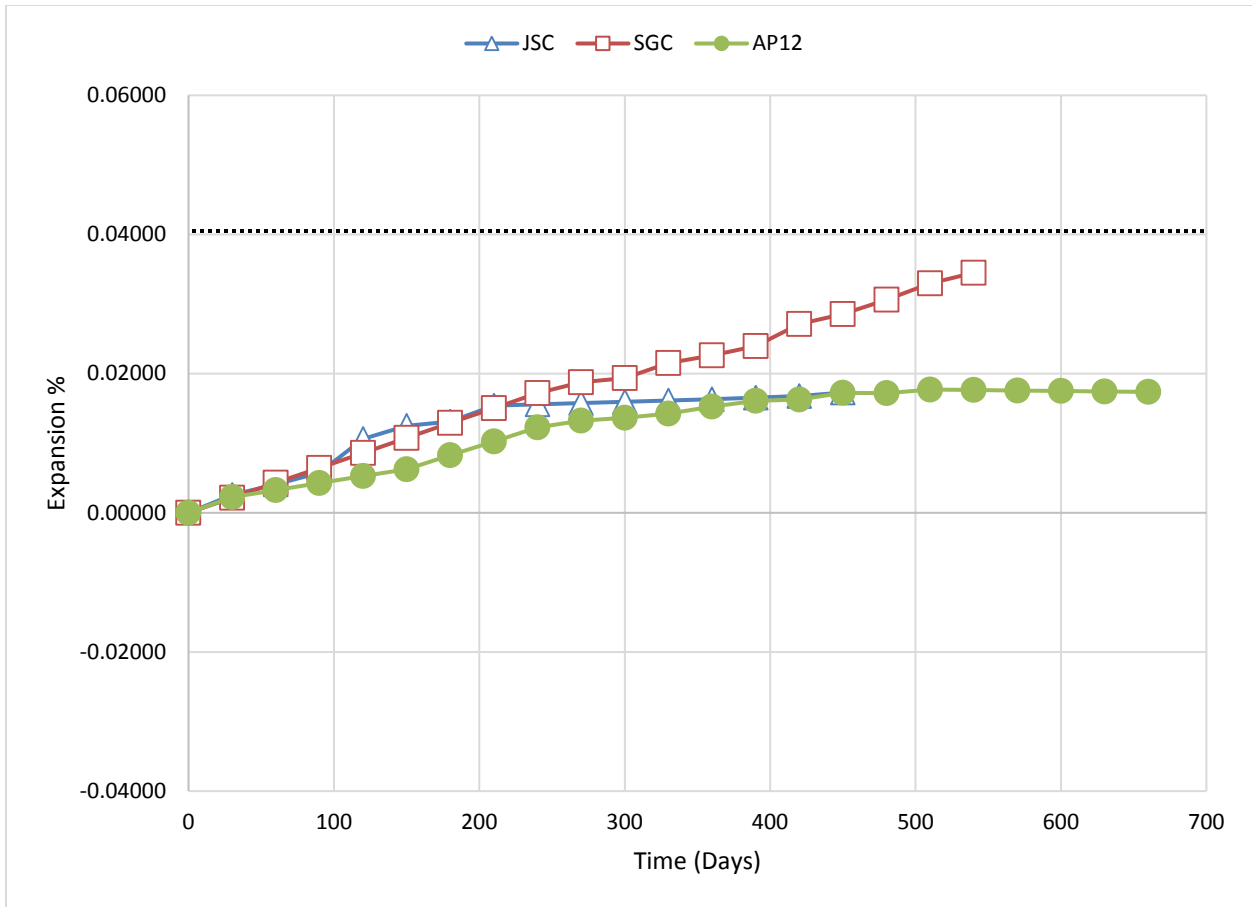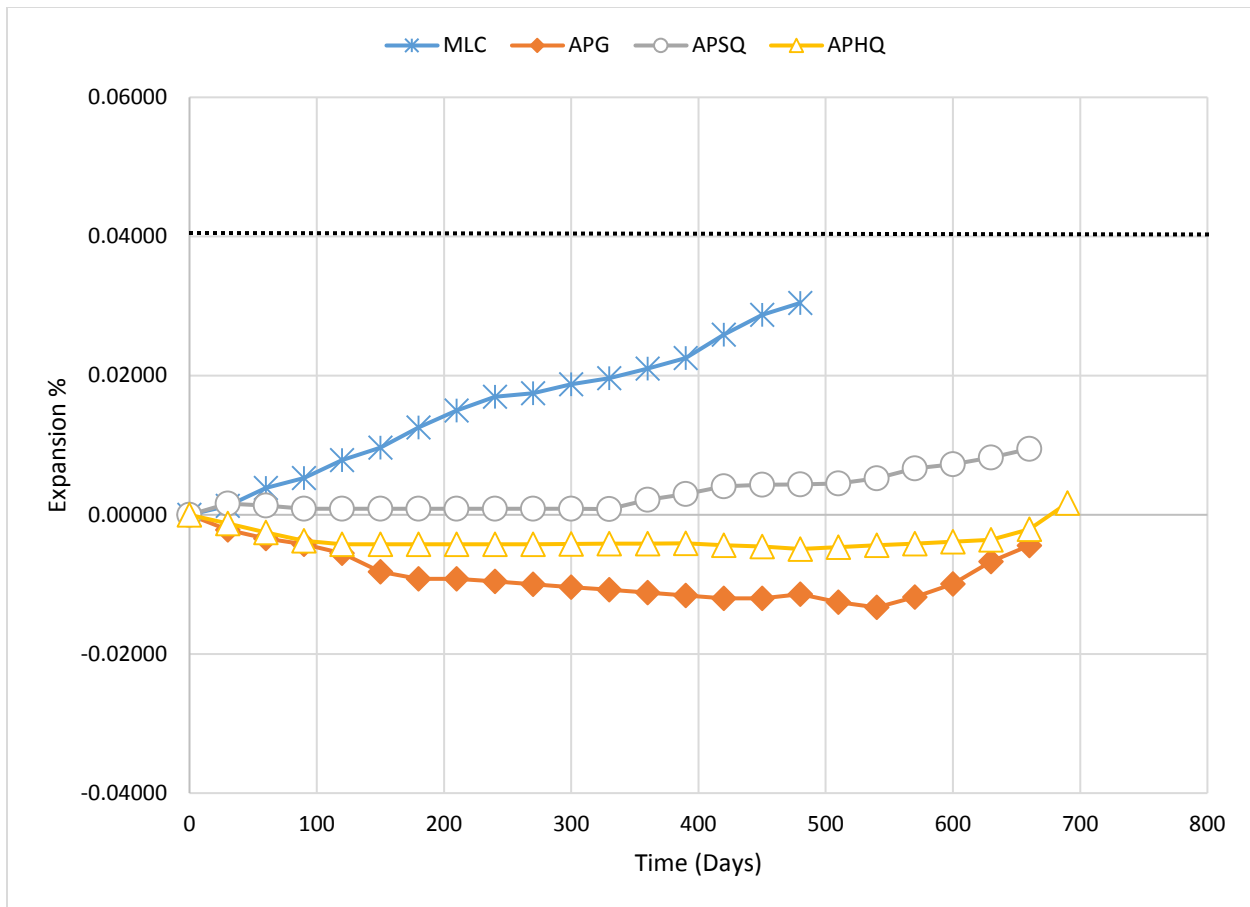
Figure 4.4.3-1: Average Strain (%) of Exposure Block Containing Limestone Aggregate

4.4.4 Exposure Blocks - Dolomite

The expansion data of the exposure blocks containing the three dolomite aggregates are shown in Figure 4.4.4-1. The three dolomite aggregates were discussed in greater detail in Section 3.2.1. The blocks are approaching two years of age and their strains are not greater than 0.04%. Based on this expansion, the dolomite aggregates included in this study would be classified as not deleteriously reactive.
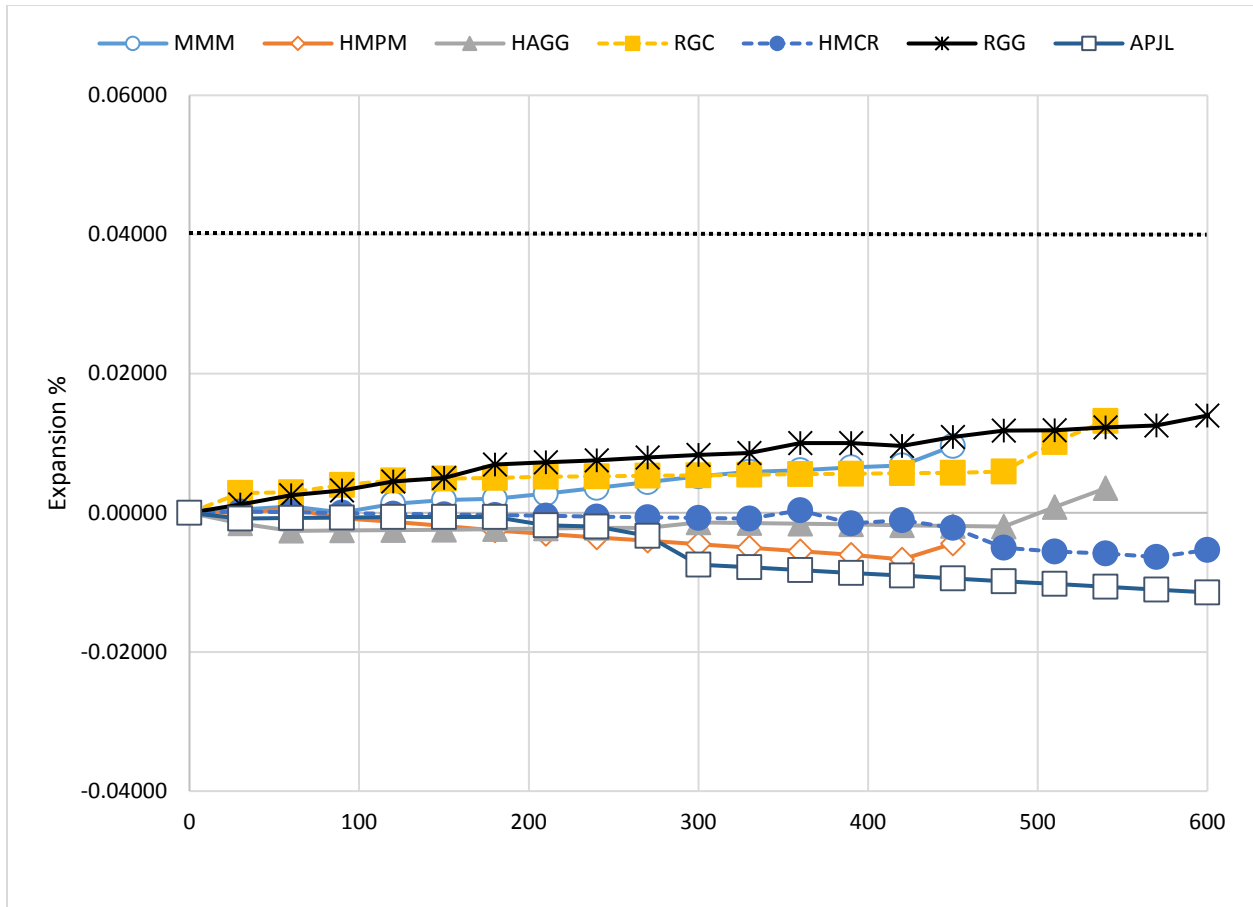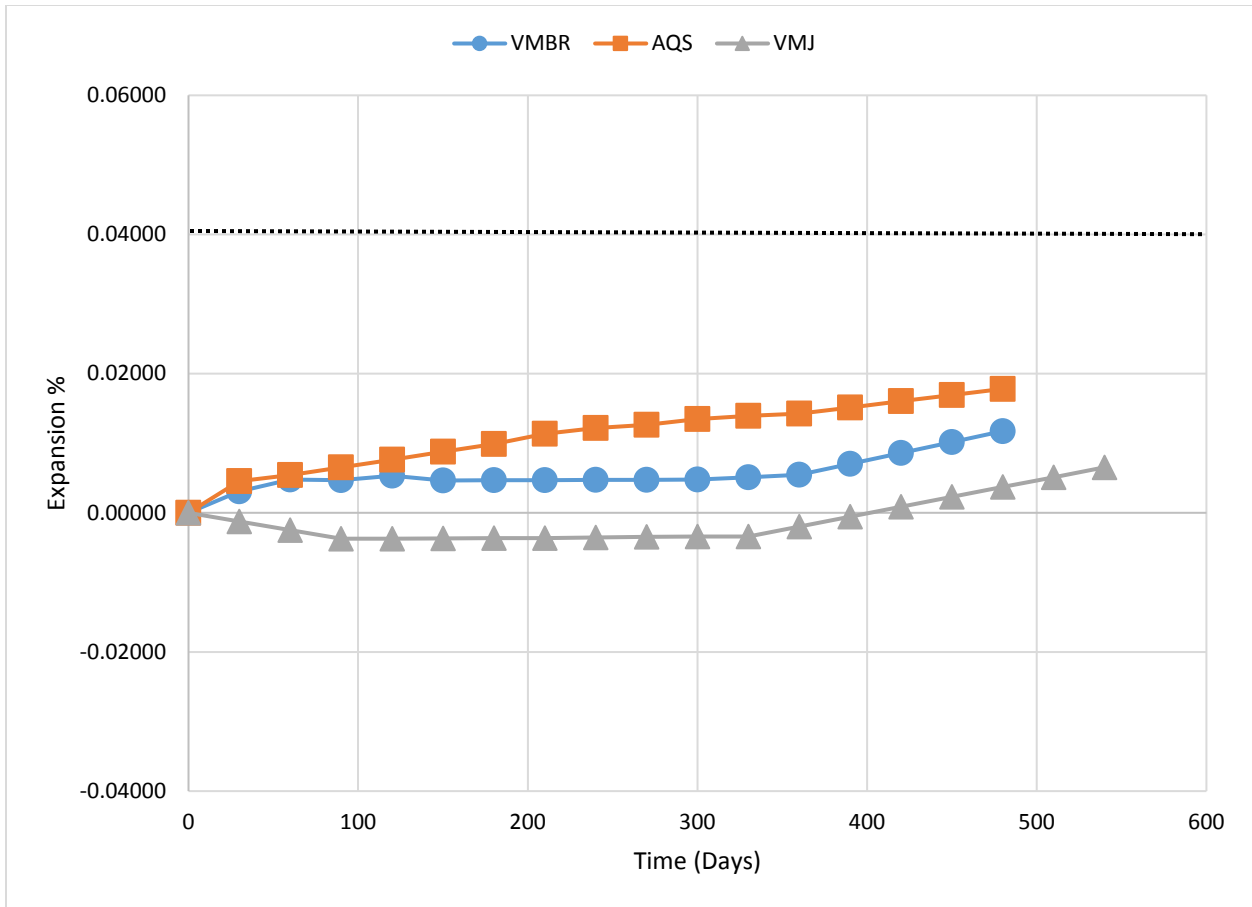
Figure 4.4.4-1: Average Strain (%) of Exposure Block Containing Dolomite Aggregate

4.4.5 Exposure Blocks - Gravel

The expansion data of the exposure blocks containing the four gravel aggregates are shown in Figure 4.4.5-1. The four gravel aggregates were discussed in greater detail in Section 3.2.1. The blocks are approaching two years of age and their strains are not greater than 0.04%. Based on this expansion, the gravel aggregates included in this study would be classified as not deleteriously reactive.
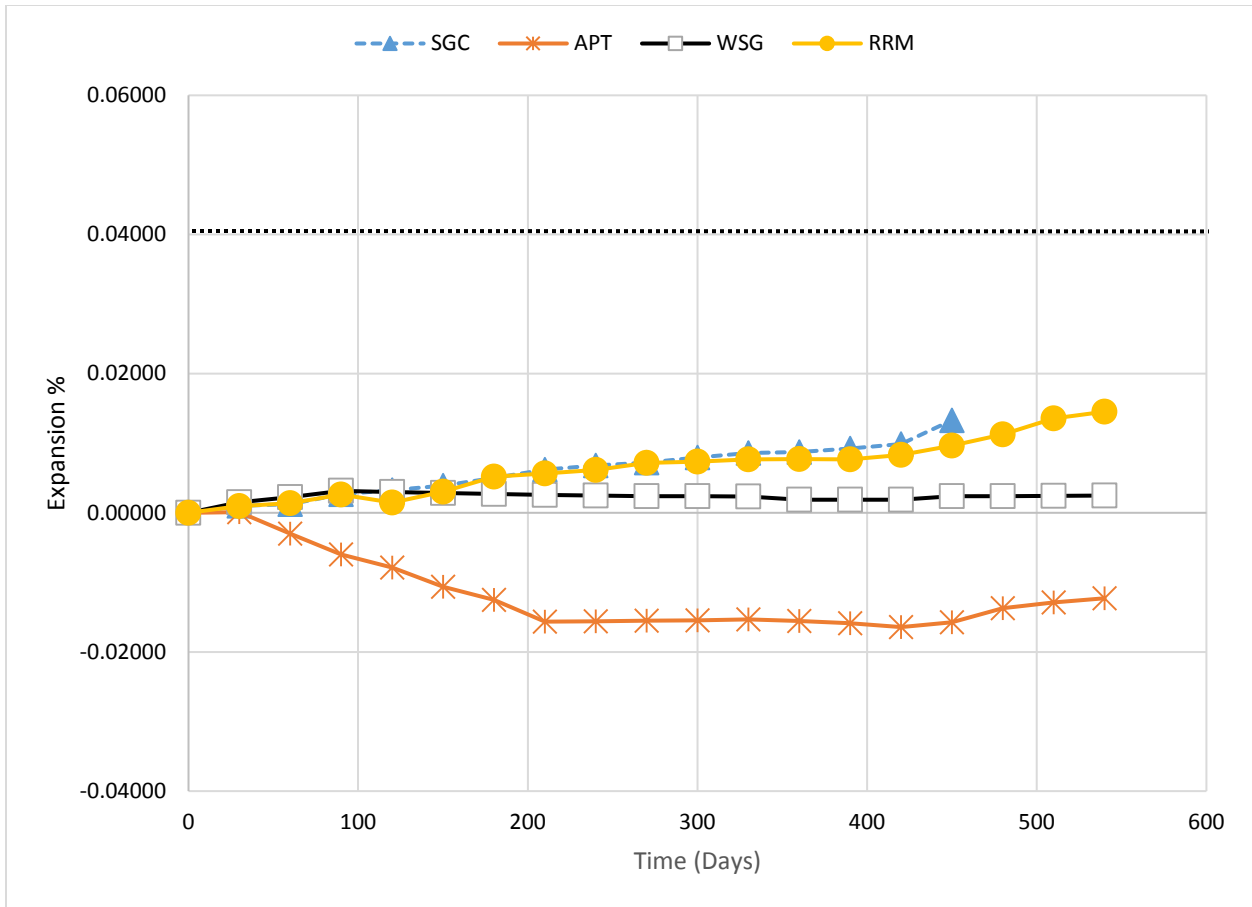
Figure 4.4.5-1: Average Strain (%) of Exposure Block Containing Gravel Aggregate

4.5 Results Summary

The following section will discuss the results from the AMBT, CPT, and the exposure blocks. The results from the findings of each test will be summarized in order to identify aggregates which were identified as deleterious from each test.

4.5.1 Deleterious AMBT

All three fine aggregate specimens, APAC Central Pools 12-14 (AP12), Standard Gravel Company (SGC), and Jeffrey Sand Company (JSC) were found to be potentially deleterious reactive based on the AMBT percent. When compared to the exposure block data there is only one specimen which exhibits potentially deleterious behavior. The Standard Gravel Company (SGC) specimen is currently at 0.034 percent at 540 days with an upward graphical trend. Assuming this trend will continue it is very likely to see potentially deleterious expansion develop. APAC Central Pools 12-14 (AP12) and Jeffrey Sand Company (JSC) have shown signs of expansion early on in the exposure blocks, but both have now plateaued at 0.017 percent strain.

4.5.2 Deleterious CPT

Four different sandstone aggregates, Hamilton Aggregates – Pryor Mt. Quarry (HMPM); Hanson Aggregate (HAGG); Hamilton Aggregates – Center Ridge (HMCR); and APAC Central Jenny Lind (APJL) exhibited deleterious expansion during the CPT. At this time there is no clear indication of deleterious potential based on exposure block data. Martin Marietta Materials (MMM), Rogers Group, Greenbrier (RGG), Rogers Group, Cabot (RGC), Hanson Aggregate (HAGG), and Hamilton Aggregates – Pryor Mt. Quarry (HMPM) have only recently started showing an upward trend indicating major expansion

The limestone aggregate specimen Midwest Lime Company (MLC) exhibited a strain 0.059 percent in the CPT. The exposure block data shows that Midwest Lime Company (MLC) has a percent strain of 0.03% at 480 days and has an upward trend. If this trend persists, the expansion will exceed 0.04% and it will be classified as potentially deleterious. Similarly, the

dolomite specimen Vulcan Materials, Judsonia (VMJ) showed a percent strain of 0.035 percent for the CPT. However, exposure block data shows a percent strain of 0.007% at 540 days, the least of the three dolomite specimens. Exposure block data shows for the last 7 months the Vulcan Materials, Judsonia (VMJ) has been in a slight expansive stage. However, it is too early to tell if this will lead to deleterious expansion.

4.5.3 Comparison of Results

Shown below in Table 4.5.3-1 is a summary of the potentially deleterious aggregates as determined by the AMBT, CPT, or the exposure blocks. For the purposes of this study, if an aggregate caused an expansion of greater than 0.04 percent in the exposure block, then it was deemed deleterious reactive. Also, for this research program, the expansion results from the exposure blocks was used to determine if an aggregate was reactive. This was done because the exposure blocks best represented real world conditions (larger specimen size, subjected to ambient conditions, representative mixture design, etc.).

As shown in Table 4.5.3-1, there were three aggregates that were identified as potentially deleterious by the AMBT. Those were three fine aggregates, AP12, SGC, and JSC. Since the expansion of these three fine aggregates were between 0.1 and 0.2 percent, further testing was recommended using the CPT. When tested using the CPT and in the exposure blocks, the expansion was less than 0.04 percent, which would classify the aggregates as not being deleterious, but the expansion of SGC shows that it will most likely exceed 0.04 percent within 2 years.

The differences between the AMBT and CPT results is supported in the literature by researchers who report that the AMBT can produce false-positive results (Thomas et al., 2006a; Ideker et al., 2012a; Touma et al., 2001). However, previous research at the University of Arkansas

has shown that Arkansas River Sand (AP12 and JSC) is reactive (Deschenes, Jr. et. al, 2017a). These differences in results on the same fine aggregate sources highlight the limitations in the current tests.

There were seven aggregates which passed the AMBT (expansion less than 0.1 percent), but failed the CPT (expansion of 0.04 percent or greater). Those aggregates are also shown in Table 4.5.3-1. When tested in exposure blocks, all of the aggregates had expansions of less than 0.04 percent at between 1 and 2 years of age, therefore the aggregates would be classified as not deleterious reactive. The outdoor exposure site test is considered the most accurate method of measuring field performance of concrete mixtures. The major limitation of the outdoor exposure site test is the extended time required to produce results, which is greater than 10 years when evaluating field performance (Ideker et. al, 2012b). Even though the expansion is less than 0.04 percent in the exposure blocks, the researchers will continue to monitor the UA blocks for the next eight years to determine if the aggregates are reactive.

Table 4.5.3-1: Potentially Deleterious Aggregate Sources

Aggregate	Abbreviation	AMBT	CPT	Exposure Block (%)
APAC Central – Pools 12 -14	AP12	Positive	Negative	0.0170
Standard Gravel Co.	SGC	Positive	Negative	0.0340
Jeffery Sand Co.	JSC	Positive	Negative	0.0170
Hamilton Agg, Pryor Mt. Quarry	HMPM	Negative	Positive	-0.0040
Hanson Aggregates – Wilton, AR	HAGG	Negative	Positive	0.0030
Rogers Group, Cabot AR	RGC	Negative	Positive	0.0130
Hamilton Aggregates Inc. – Center Ridge, AR	HMCR	Negative	Positive	-0.0050
APAC Central – Jenny Lind, AR	APJL	Negative	Positive	-0.0110
Midwest Lime Co.	MLC	Negative	Positive	0.0300
Vulcan Materials Company - Judsonia, AR	VMJ	Negative	Positive	0.0060

CHAPTER 5: CONCLUSIONS & RECOMMENDATIONS

5.1 Introduction

The objective of this research project was to catalog potentially reactive aggregate obtained from sources approved by the Arkansas Department of Transportation. To achieve this goal, the effectiveness of laboratory testing (AMBT, CPT) was compared to actual field conditions (exposure blocks) and from this data effectively identify the deleterious potential of frequently used aggregates in the state of Arkansas.

5.2 Conclusions

The results of the AMBT show three potentially reactive fine aggregates from sources APAC Central Pools 12-14, Standard Gravel Company, and Jeffrey Sand Company. The results from the CPT show that there are five potentially reactive sandstone aggregates from Hamilton Aggregates – Pryor Mt. Quarry, Hanson Aggregate, Rogers Group, Cabot, Hamilton Aggregates – Center Ridge, and APAC Central Jenny Lind. Also there is one potentially reactive limestone aggregate and one dolomite aggregate, from Midwest Lime Company and Vulcan Materials, Judsonia, respectfully. However, the expansion data from the exposure blocks show that all aggregates are not reactive at 1 to 2 years of age.

5.3 Recommendations

The results from either the AMBT or CPT laboratory investigation suggest there are ten potentially deleteriously reactive aggregate sources. However, the exposure block expansion rates show that none of the aggregates are reactive. As previously stated, the outdoor exposure site test

is considered the most accurate method of measuring field performance of concrete mixtures. The major limitation of the outdoor exposure site test is the extended time required to produce results, which is greater than 10 years when evaluating field performance (Ideker et. al, 2012b). Even though the expansion is less than 0.04 percent in the exposure blocks, the researchers will continue to monitor the UA blocks for the next eight years to determine if the aggregates are reactive. To prevent future occurrences of ASR, it is recommended that concrete mixtures, using the aggregates examined in the study, contain 30 percent Class C fly ash (Deschenes Jr. and Hale, 2017b). This has been shown to prevent ASR from occurring.

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