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16. Abstract This research project involved an experimental evaluation of the sulfate resistance of various concretes and mortars for the purpose of establishing performance-based specifications for the durability of concrete against sulfate attack. The research is the test program portion of a long-term project that aims to establish performance-based specifications for durability for concrete mix designs for the Texas Department of Transportation. The test program described in this report consisted of two phases. The first phase was an investigation of the potential use of the standardized test, ASTM C1012-95, as a performance test. Plain portland cement mortars and mortars with cement-mineral admixture combinations were tested using three ASTM C150 types of cement, two types of fly ash, and a ground granulated blast furnace slag. The second phase involved evaluating experimentally the effect of permeability on the sulfate resistance of concrete for the purpose of determining the proper approach for specifying permeability for sulfate resistance. Concrete permeability was varied by testing different types of cement and cement-mineral admixture combinations, by using different water-to-cementitious material ratios for each cement combination, and by using various curing procedures. In the first phase mortar bars and in the second phase concrete cylinders were immersed in sulfate solution to simulate sulfate exposure.			
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**EVALUATING PERFORMANCE-BASED TEST AND SPECIFICATIONS FOR
SULFATE RESISTANCE IN CONCRETE**

by

John B. Stephens and Ramón L. Carrasquillo

Research Report Number 1706-3

Research Project 0-1706

Optimization of Concrete Mix Designs for Durability

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

by the

CENTER FOR TRANSPORTATION RESEARCH

Bureau of Engineering Research

THE UNIVERSITY OF TEXAS AT AUSTIN

May 1999

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Chapter One: Introduction

1.1 Purpose of Work

This research involved an experimental evaluation of the resistance of various mortar and concrete mixes to sulfate exposure for the purpose of establishing performance-based specifications for durability. The first portion of testing involved the examination of a performance test for evaluating mortars' resistance to sulfate attack. The accuracy and consistency of the test as well as its corresponding performance criteria were investigated. The second portion of testing involved an investigation of the influence concrete permeability has on sulfate resistance. Alternatives for specifying permeability were examined.

The test program is part of a long-term project that involves the development of performance-based specifications for durable concrete for the Texas Department of Transportation (TxDOT). In previous research, new specifications for evaluating the sulfate resistance of concrete were proposed. This research investigated the accuracy of the performance test and performance criteria recommended in those proposals to ensure the specifications provide an effective guideline for evaluating the long-term durability of concrete against sulfate attack.

1.2 Background

In a time when a significant amount of funds in the transportation industry are going toward repairing, retrofitting, and rehabilitating current concrete infrastructure, it has become critical that engineers emphasize durability and long-term performance in all new construction projects. Guidelines need to be developed to ensure that new materials, technologies, test methods, and construction practices can be utilized to produce high quality, long-lasting concrete structures. Currently, engineers are limited by the parameters and guidelines that were developed several years ago. Specifications have not incorporated the new technologies that have come about in the concrete industry during the last few decades. TxDOT recognized this limitation and, as a result, began a research effort to develop a new set of guidelines and specifications for producing durable concrete.

In developing new specifications, TxDOT wanted a change in the approach taken towards writing guidelines for durability of concrete mix designs. Current TxDOT specifications for concrete construction do not address the properties of concrete that most directly affect durability. Instead, these relevant properties are addressed by indirect means. For example, the permeability of concrete is considered to be one of the most important properties affecting concrete durability. Under today's specifications, if an engineer needed a low permeability concrete to ensure durability in a severe environment, this need would be addressed by keeping the mix design water-to-cementitious material ratio below a specified maximum value. The specified maximum water-to-cementitious material ratios were derived nearly 40 years ago from a correlation that was established between the water-to-cement ratio and permeability. The problem with using this correlation in current specifications is that it does not acknowledge all the factors that affect concrete permeability in today's construction industry. Improved cements, chemical and mineral admixtures, and construction techniques are now available so that low permeability can be accomplished in many ways.

The goal of the new specifications is to address durability needs directly. If a certain level of permeability is required, specifications will provide a maximum allowable permeability value. Using a specified performance test, engineers can measure the concrete permeability and verify that it meets the job requirements. TxDOT aims to eliminate the practice of achieving adequate permeability through the means of specifying indirect limits such as maximum water-to-cementitious material ratios. These new types of specifications are referred to as *performance-based specifications*. The specifications give engineers the freedom to utilize any technologies or resources available to produce a concrete that meets or exceeds durability needs for a given job condition. All the specifications require is that the concrete be shown through testing to have the properties necessary for adequate performance.

1.3 Project Description

This research is part of a project funded by the Texas Department of Transportation titled "Optimization of Concrete Mix Designs for Durability." The project's ultimate goal is the development of performance-based specifications for concrete structures and other

concrete applications for TxDOT. Initial research for this project involved the formation of a durability model that outlined the relevant concrete properties that affect each aspect of concrete durability and selected test procedures for evaluating those properties. Through a literature review of the state of the art in the concrete industry, a durability model was developed addressing five aspects of concrete durability: freezing and thawing, sulfate attack, alkali-silica reaction, abrasion, and corrosion of reinforcing steel.

The next stage in the project involved validating and refining recommendations presented in the durability model through an experimental investigation. For this portion of the project, the test program for this report was developed. The focus of the experimental investigation was on the durability of concrete against sulfate attack, as this area was one where a relatively new test was proposed and where the effects of certain properties were unclear. Results and conclusions from this test program should lead to the refinement of the durability model and, as the final stage, completion of the performance-based specifications.

1.4 Objectives

In developing the durability model for performance-based specifications, the researchers noted that one aspect of durability for which investigation beyond a state-of-the-art literature review appeared to be needed involved the issue of sulfate attack. The goal of this research is to perform this continued investigation in the form of a two-phase testing program.

The first phase of the test program involved the evaluation of the American Society for Testing and Materials (ASTM) standardized test, ASTM C1012-95 (Ref 1), titled “Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution.” This test, with its wide range of applicability and relatively short duration, has proven to have excellent potential as a performance test for evaluating the ability of mortars to resist sulfate attack. The test, however, is relatively new and is not yet widely used in the industry. The objective of this first phase is to examine the reliability of the ASTM C1012-95 test and its corresponding performance criteria. A wide range of cement types and cement-mineral admixture combinations will be tested to determine the test’s

ability to identify mortar properties affecting sulfate resistance. The ultimate goal is to prove if ASTM C1012-95 is an effective test that can be used with confidence by TxDOT as a performance evaluation test for sulfate attack.

The second phase of the test program is an evaluation of the effect of permeability on the concrete's ability to resist sulfate attack. While it is intuitive to believe permeability has an effect on sulfate resistance as it measures the ability of sulfate ions to penetrate into the concrete, it is not clear as to the impact the property has on sulfate resistance. Current literature is inconclusive. Some research has indicated low permeability is not capable of fully preventing sulfate attack in concrete; however, current specifications still require low permeability concrete for concrete in sulfate environments by means of limiting the maximum water-to-cementitious material ratios. The goal of this testing phase is to evaluate and compare the influence of concrete permeability and the chemistry of the cementitious materials on the sulfate resistance of concrete. Information from this experimentation will be used to decide what requirements need to be met for permeability concerning sulfate attack and how these requirements can be incorporated into the performance-based specifications.

1.5 Scope

The first phase of the testing program reported herein, referred to as Phase 1 throughout the report, involved the testing of several different mortar mixes using the standardized test, ASTM C1012-95, "Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution." Four different cements, all commercially produced in the state of Texas, were evaluated; these included one ASTM C150 (Ref 2) Type I cement, two ASTM C150 Type I-II cements, and one ASTM C150 Type V cement. Mortar mixes were also made with mineral admixtures as each cement was combined with three different types of mineral admixtures. One ASTM Class F (Ref 3) fly ash, one ASTM Class C (Ref 3) fly ash, and one ground granulated blast furnace slag (GGBFS) was added in various percent volumetric replacement levels. Mortar bars were made and soaked in a sulfate solution. Expansion measurements were taken and compared to expansion criteria recommended from experience.

The second phase, referred to as Phase 2 throughout the report, involved the evaluation of several concrete mixes using the United States Bureau of Reclamations test, USBR 4908 Method B (Ref 4), titled “ Procedure for Length Change of Hardened Concrete Exposed to Alkali Sulfates.” The test involved soaking concrete cylinders in a sulfate solution and measuring the length and mass change of the specimens. The permeabilities of the concrete mixes were measured using the standard rapid permeability test, ASTM C1202-94 (Ref 5), titled “Standard Test Method for Electrical Indication of Concrete’s Ability to Resist Chloride Ion Penetration.” The same four cements used in Phase 1 (except one of the Type I-II cements) were tested. For some concrete mixes, the two types of fly ash from Phase 1 were blended with the Type I-II cement. For each cement or cement-fly ash combination at least three batches were made with different water-to-cementitious material ratios to vary the permeability of the mixes. Also, specimens from each batch were cured under different temperature conditions to vary the permeability of the hardened concrete within the batches. Owing to the long-term nature of this portion of the testing, final results for Phase 2 will not be presented in this report. Only mix designs, compressive strengths, permeability data, and initial expansion and mass changes will be reported and discussed. Complete results and a final analysis of the implications of this testing will be reported at a later time.

Future work for this project will involve the finalization of Phase 2 results and the drafting of performance-based specifications for the Texas Department of Transportation.

Chapter Two: Literature Review

2.1 Introduction

Before presenting the details of the testing program, it is important to review the state of the art regarding sulfate attack and what is currently being done to address the problem of sulfate attack in concrete structures. This chapter discusses the mechanisms of sulfate attack; the effects of relevant concrete properties on sulfate resistance; current specifications and guidelines for producing sulfate resistant concrete; available tests for examining or predicting concrete or mortar sulfate resistance; and finally, proposed performance-based specifications. Much of the information presented herein was derived from the state-of-the art literature review performed in an earlier stage of this project (Ref 6).

2.2 Mechanism of Sulfate Attack

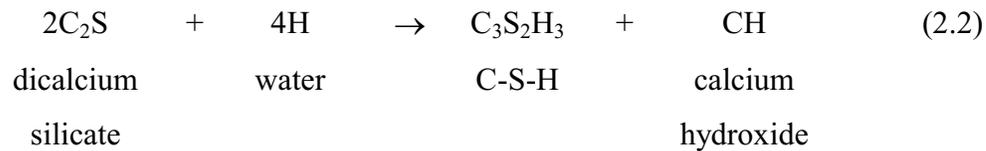
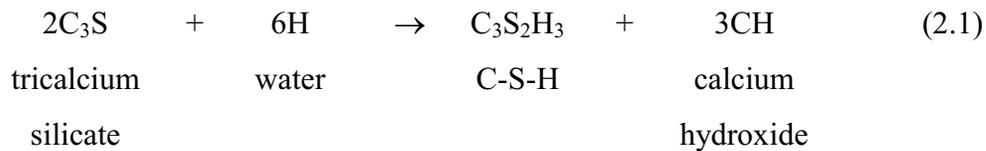
In previous research for the Texas Department of Transportation, Tikalsky and Carrasquillo defined sulfate attack as a series of reactions that may occur in hardened concrete in the presence of sulfate ions (Ref 7). The products of these reactions occupy a greater volume than the host reactants (Ref 7), and thus cause an internal expansion of the cement paste and localized tensile stresses in the hardened concrete. Sulfate attack ultimately manifests itself in the form of cracking, spalling, and mass loss, all of which lead to loss of cross section or complete deterioration of the affected concrete elements. Naturally occurring sulfates of sodium, potassium, calcium, and magnesium, which may be found in soils or dissolved in groundwater, can attack concrete. These sulfates are always present in seawater (Ref 8).

Most experts believe that the formation of ettringite in hardened concrete is the principal cause of sulfate attack. Other hypotheses regarding the mechanisms of sulfate attack include deterioration by magnesium ions from magnesium sulfate, the swelling of already-formed ettringite crystals as they absorb water, the formation of gypsum, and the scaling of concrete as a result of sulfate salt crystallization. This review will focus on what is

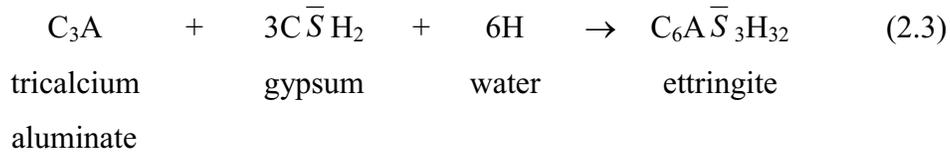
considered by most to be the primary mechanism of sulfate attack, namely, the formation of ettringite.

In one form, the process of ettringite formation involves a product of cement hydration, monosulfoaluminate, transforming into ettringite in the presence of excess sulfate ions in hardened concrete. The volume of ettringite formed is approximately twice that of the original monosulfoaluminate (Ref 7). A better understanding of these two phases, monosulfoaluminate and ettringite, and the reactions described can be obtained by reviewing the process of cement hydration.

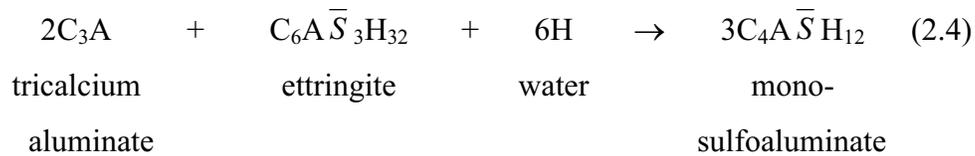
The five primary compounds that constitute portland cement are tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), tetracalcium aluminoferrite (C_4AF), and a sulfate hydrate compound called gypsum ($C\bar{S}H_2$) that is added to portland cement clinkers to control flash setting. Tricalcium silicate and dicalcium silicate each react with water to form calcium silicate hydrate (C-S-H) and calcium hydroxide as shown in Equations 2.1 and 2.2 (Ref 7). C-S-H is the primary binding component of hydrated portland cement. Calcium hydroxide is a water-soluble by-product that has no cementitious value.



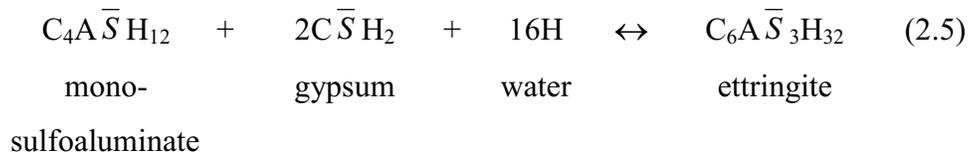
A secondary binding component is formed from the hydration of tricalcium aluminate and gypsum. The C_3A and gypsum combine to form ettringite as shown in Equation 2.3 (Ref 7). While this formation of ettringite is an expansive reaction, the expansion is not harmful in fresh concrete because the concrete is still plastic and can accommodate volume changes.



The ettringite formed is stable only in high concentrations of sulfate. As hydration progresses, the concentration of sulfate drops as gypsum is consumed. The ettringite becomes unstable with this decrease in sulfate concentration and reacts with the remaining C_3A to form monosulfoaluminate. This reaction is shown in Equation 2.4 (Ref 7). Monosulfoaluminate is typically a dominant portion of the aluminate crystalline phases of the hydrated cement matrix (Ref 7).



The critical reaction that defines sulfate attack occurs when the concrete is in its hardened state. If hardened concrete is exposed to an external source of soluble sulfate ions, the concentration of sulfates in the concrete pore water may increase. As sulfate ion concentration increases, the monosulfoaluminate becomes unstable and reverts back to ettringite as shown in Equation 2.5 (Ref 7).



This reaction is the one that most commonly causes problems because the ettringite phase that is formed occupies twice the volume of the monosulfoaluminate phase. When the void spaces in the paste cannot accommodate any further expansion, internal tensile strains

may cause cracking of the hardened concrete. This leads to the loss of section and deterioration that is typical of sulfate attack in concrete.

2.3 Factors Affecting Concrete Resistance to Sulfate Attack

Several factors affect the resistance of concrete to sulfate attack. These factors include the chemistry of the cementitious material, the permeability of the concrete, the concentration of external sources of sulfate ions, and the severity of the concrete exposure to the sulfate environment. The last two factors, sulfate concentration and severity of exposure, are both generally decided when the location and type of concrete application are chosen. Based on the environment established by these factors, the engineer must control sulfate attack by controlling the *first* two factors — the properties of the concrete. This section reviews three major concrete properties that influence concrete sulfate resistance. These properties are the chemistry of the portland cement, the permeability of the concrete, and the chemistry and replacement level of mineral admixtures. These are the properties that will be evaluated in this report's testing program.

2.3.1 Chemistry of Portland Cement

The concrete characteristic with the greatest impact on sulfate resistance is generally considered to be the chemistry of the portland cement as this has a direct impact on the chemical reactions that cause sulfate attack. The five primary components of portland cement and their contribution to the process of sulfate attack were reviewed in Section 2.2. Equations 2.3, 2.4, and 2.5 show that the tricalcium aluminate (C_3A) and gypsum ($C\bar{S}H_2$) compounds have the largest impact on sulfate attack because they are directly involved in the formation of ettringite and monosulfoaluminate in the fresh concrete and the formation of ettringite in the hardened concrete. Most research today identifies tricalcium aluminate as the primary measurable component of portland cement that influences sulfate resistance in concrete. The C_3A controls the amount of monosulfoaluminate that is formed in fresh concrete and thus the amount of monosulfoaluminate available to form ettringite in hardened

concrete resulting in expansive internal volume changes and the damage that characterizes sulfate attack.

In 1949, the Portland Cement Association (PCA) published a report, *Long-Time Study of Cement Performance in Concrete*, that evaluated two factors influencing sulfate resistance, cement content, and C_3A content in cement (Ref 9). PCA concluded that for concrete mixes with intermediate or low cement contents, limiting the maximum C_3A content of the cement improved resistance to sulfate attack. The research showed that concrete made with cements having C_3A contents greater than 7 percent performed poorly while concrete with cements having C_3A contents of 7 percent or lower performed well. Further studies were conducted to distinguish between good and excellent performance of concrete as a function of the cement (Ref 10). PCA research showed that for concrete made with rich cement contents, mixes containing cements with 5.5 percent C_3A as corrected for minor oxides exhibited better performance than other high cement content concrete. These PCA studies started to establish values for limiting the C_3A content of cements for use in various levels of sulfate exposure.

In the concrete industry, each cement plant produces a portland cement with a different relative proportion of its five primary compounds. ASTM Standard Specification C150-94 (Ref 2) classifies these portland cements into five types that are recognized throughout the industry. Each type has required chemical and physical characteristics. The need for sulfate resistance in a cement is acknowledged in ASTM C150-94 via the establishment of maximum C_3A contents for each cement type. The limits address the fact that the C_3A content is the critical property of the cement chemistry that affects sulfate resistance in concrete. The ASTM C150-94 limits are shown in Table 2.1.

The three cement types that are evaluated in this test program and that are generally considered when choosing a cement for concrete in a sulfate environment are Type I, Type II, and Type V portland cement. Type III portland cement is a high early strength cement. With its 15 percent maximum C_3A content, it is considered unsuitable for sulfate environments. Type IV portland cement is a low heat of hydration cement used only in cases where the generation of internal heat is a concern, such as in mass concrete. While its

maximum C_3A content is acceptable, its limited availability and slow rate of strength gain precludes it from use in most sulfate environments.

Table 2.1 – Maximum C_3A Contents for Portland Cement (Ref 2)

ASTM C150 Cement Type	Maximum C_3A Content, %
I	—
II	8
III	15
IV	7
V	5

Type I portland cement is a general-use cement that is widely available. Because ASTM C150-94 does not limit the C_3A content of this cement, most Type I cements have high C_3A contents and are thus not suitable for moderate or severe sulfate environments. Combinations of the cement with mineral admixtures, however, are still being considered as an option for concrete in sulfate environments. Type II portland cements or Type I-II portland cements are considered to be moderately sulfate resistant cements. ASTM C150-94 limits the C_3A content of these cements to 8 percent. This limit is similar to the 7 percent plateau for good performance that the PCA research established earlier. Type II cements are generally used by the Texas Department of Transportation in areas of potential sulfate attack. Type V portland cements are considered to be highly sulfate resistant cements. The ASTM limit of 5 percent for the Type V cement's C_3A content is similar to the 5.5 percent value determined by PCA for excellent performance. Owing to its low availability, especially in Texas, and thus high cost, Type V cements are generally reserved for the extreme cases of a severe or very severe sulfate environment.

2.3.2 Concrete Permeability

When sulfate attack was first identified in 1908 by the United States Bureau of Reclamations (USBR), the only means of controlling the attack was by producing low

permeability concrete (Ref 11). In today's industry low permeability concrete can be produced in several ways. These methods include reducing the water content and/or increasing the cement content such that the water-to-cementitious material ratio is reduced, selecting proper materials and mix proportions, replacing a percentage of cement with a mineral admixture, using air entrainment, adequately consolidating the concrete during placement, and providing proper and effective curing. The goal of using low permeability concrete is to minimize the penetration of sulfate ions into the concrete. Keeping the sulfate ion concentration in the hardened concrete low prevents the deleterious formation of ettringite from monosulfoaluminate described in Equation 2.5.

In previous research for the Texas Department of Transportation, Freeman made some observations about the true effect of permeability on the ability of concrete to resist sulfate attack (Ref 12). In his testing, Freeman measured damage resulting from sulfate attack by both linear expansion and mass loss of concrete cylinders being soaked in sulfate solution. For most results, the combination of linear expansion and mass loss by spalling of the specimens revealed poor performance. However, Freeman did observe that for some of the low permeability specimens, little expansion occurred but there was some mass loss measured. From visual observation of the specimens, Freeman noticed that while the low permeability specimens were experiencing no significant linear expansion from sulfate attack, the specimens did sustain damage from surface scaling. Freeman's research revealed that low permeability in concrete does not necessarily make it immune to sulfate attack as damage may still occur in the form of surface scaling. Freeman's work confirmed earlier research performed by Tikalsky (Refs 7, 13) for TxDOT, which showed that low permeability concrete may suffer from sulfate attack despite the appearance of high quality. Tikalsky concluded that a substantial reduction in permeability does not necessarily render concrete resistant to deterioration if the chemistry of that concrete would otherwise be susceptible to attack (Ref 13).

While most researchers still agree that permeability is a property that directly affects the durability of concrete against sulfate attack, it is unclear as to the level of impact permeability has on sulfate attack. Freeman's and Tikalsky's research both make it evident

that while low permeability is a beneficial concrete attribute in resisting sulfate attack, permeability alone cannot prevent sulfate attack in concrete. The purpose of Phase 2 of the test program discussed herein is to provide a better understanding of how necessary or unnecessary low permeability is for protecting concrete against sulfate attack. Both Tikalsky's and Freeman's work were focused on studying the effects of fly ash on sulfate resistance, and the discoveries related to permeability were secondary conclusions derived from the research. Phase 2 provides a study where the primary focus is the evaluation of the effect permeability has on the sulfate resistance of concrete with varying cementitious chemistries.

2.3.3 *Mineral Admixtures*

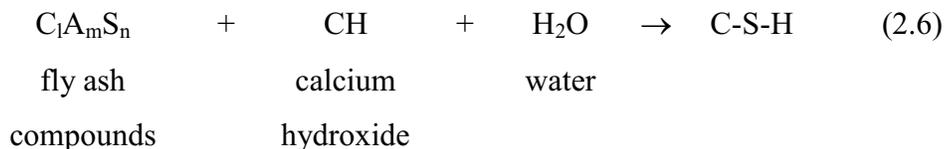
The potential benefit of any mineral admixture must be individually determined as admixtures have shown to increase, decrease, or have no affect on the sulfate resistance of concrete. The obvious benefits of mineral admixtures are the reduction in concrete permeability and the replacement of the portland cement. Lowering the permeability slows the penetration of sulfate ions into hardened concrete while replacing the portland cement reduces the presence of compounds such as C_3A that cause ettringite formation. The mineral admixtures most frequently examined for use in sulfate environments include fly ash, silica fume, and blast furnace slag. This section focuses on fly ash and ground granulated blast furnace slag, the two types of mineral admixtures evaluated in the test program for this report.

The effect of fly ash, a by-product from the burning of coal, on sulfate attack has been widely researched. Tikalsky's and Freeman's research at The University of Texas at Austin studied the effects of fly ash on sulfate attack in detail as twenty-four fly ashes from around the United States were evaluated in each study (Refs 7, 12). The findings of these studies are reviewed in the following paragraphs.

The chemical composition of a fly ash determines its ability to increase, decrease, or not change the resistance of concrete to sulfate attack. The five chemical and mineralogical components of fly ash that affect sulfate resistance are calcium, alumina, iron oxide, silica,

and sulfate. The calcium content is the most important of these five components. Low calcium, pozzolanic fly ashes are typically designated as ASTM Class F fly ashes. These fly ashes are described as pozzolanic because they primarily hydrate by reacting with the calcium hydroxide formed from the portland cement's calcium silicates as shown in Equation 2.1 and 2.2. High calcium, pozzolanic and cementitious fly ashes are typically designated as ASTM Class C fly ashes. These fly ashes are cementitious because they can provide their own source of calcium and thus hydrate independent of the portland cement. In general, low calcium fly ashes have been shown to improve sulfate resistance while high calcium fly ashes have either shown little improvement or decreased sulfate resistance.

Low calcium, ASTM Class F fly ashes chemically improve the sulfate resistance of concrete by consuming calcium hydroxide and thus reduce the potential for the formation of ettringite. Calcium hydroxide is the main source of calcium in concrete for the formation of ettringite. In the presence of sulfates, calcium hydroxide converts to gypsum in the hardened concrete. This gypsum then reacts with the monosulfoaluminate to form the expansive ettringite as shown in Equation 2.5. Low calcium fly ashes reduce the availability of calcium hydroxide because the fly ash reacts with it in the pozzolanic reaction shown in Equation 2.6 (Ref 7). The reaction forms the stable secondary binding compound, C-S-H. The reduced amount of calcium hydroxide resulting from this reaction lowers the amount of gypsum that can be produced in the hardened concrete and thus reduces the potential for the formation of ettringite. This process is often referred to as *pozzolanic consumption*.



High calcium, ASTM Class C fly ashes also provide pozzolanic consumption; however, this benefit is often counteracted by the large amounts of calcium the fly ash contributes to the concrete. The usual result of using these fly ashes is either little improvement or even reductions in the sulfate resistance of the concrete.

While the calcium content of fly ashes is a major factor influencing the effect of fly ashes on a concrete's sulfate resistance, the alumina, iron oxide, silica, and sulfate compounds in fly ash also contribute. Alumina may be present as part of the glassy phases or in several crystalline phases. The glassy-phase aluminas typically negatively affect sulfate resistance as they react with calcium hydroxide to form C-A-H, which is susceptible to attack in sulfate environments. High calcium fly ashes contain more glassy-phase aluminas than low calcium fly ashes because the amount of alumina in the glassy phases of fly ash is generally proportional to the amount of analytic calcium in the fly ash (Ref 7). The effect of crystalline-phase aluminas on sulfate resistance depends on their form. In low calcium fly ash, alumina often takes the crystalline form of mullite, which is stable in sulfate environments. In high calcium fly ashes, alumina may be present in the crystalline form melilite, C_3A , or $C_4A_3\bar{S}$. Melilite is stable in sulfate environments, but the C_3A and $C_4A_3\bar{S}$ in some high calcium fly ashes may contribute to the formation of ettringite.

Researchers have been divided on the effect of iron oxide in fly ash on sulfate resistance. Kalousek explained that iron oxide can exist in three phases, each contributing differently to sulfate resistance (Ref 14). Reactive phases of iron oxide improve sulfate resistance by forming nonexpansive, iron-rich ettringite and reducing the potential for the formation of expansive ettringite. By comparison, the crystalline phase of iron oxide typically has no effect on sulfate resistance. Finally, the glassy phase of iron oxide, if greater than 10 percent of the total iron oxide, may form calcium ferrite hydrate that is susceptible to sulfate attack.

During the pozzolanic reaction shown in Equation 2.6, the silica in fly ash reacts with calcium hydroxide, forming a stronger and less permeable concrete. Also, the slower pozzolanic reaction may surround the reactive aluminate compounds from the faster cement hydration reactions with C-S-H. C-S-H is stable in sulfate environments and thus prevents the formation of ettringite by isolating the aluminate compounds from sulfate ions.

Sulfate, measured as sulfur trioxide, is the final component of fly ash that affects sulfate resistance. Large quantities of sulfate in fly ash may be beneficial to the sulfate resistance of concrete because the concrete is supersulfated and the sulfates promote the

formation of ettringite while the concrete is still plastic. Fly ashes with low or moderate quantities of sulfate have little affect on the sulfate resistance of concrete (Ref 7).

Several methods have been offered to predict the effect of fly ashes on the sulfate resistance of concrete. Dunstan proposed a sulfate resistance factor, R, to determine the effectiveness of different fly ashes (Refs 15, 16). The factor is based on the chemistry of the fly ash and is defined below:

$$R = \frac{(CaO - 5)}{Fe_2O_3} \quad (2.7)$$

The proportions of calcium oxide (CaO) and ferric oxide (Fe₂O₃) in Equation 2.7 are expressed as percent by mass. The ability of the fly ash to resist sulfate attack based on its R factor can be determined using Table 2.2.

Table 2.2 – R Factor for Sulfate Resistance of Concrete with Fly Ash (Ref 16)

R factor	Sulfate Resistance
< 0.75	Greatly improved
0.75 to 1.5	Moderately improved
1.5 to 3.0	No significant change
> 3.0	Reduced

Dunstan’s R factor provides a conservative method for selecting fly ashes for use in concrete in a sulfate environment. The R factor has been shown to be too conservative in some cases as some fly ashes that did not meet the R factor limit still performed well in tests (Refs 17, 18). Tikalsky and Carrasquillo (Ref 19) recommended an alternative method for selecting fly ashes for producing sulfate resistant concrete. The recommendations are as follows:

1. Fly ash meeting ASTM C 618-94a (Ref 3) and containing less than 10 percent CaO may be used to improve sulfate resistance of concrete.
2. Fly ash meeting ASTM C 618-94a and containing between 10 percent and 25 percent CaO should be further examined using USBR 4908-86 (Ref 4) or ASTM C 1012-95 (Ref 1). The reactivity of the glass phase determined by x-ray diffraction (Ref 20) and bulk composition analysis (Ref 21) should be determined from the ternary diagram shown in Figure 2.1. The fly ash is likely to improve sulfate resistance if the glassy portion of the ash lies in the mullite or upper half of the anorthite fields. Fly ash with the glassy portion in the gehlenite field or lower half of the anorthite field may decrease sulfate resistance.
3. Fly ash containing more than 25 percent CaO may not be used in concrete exposed to sulfate.

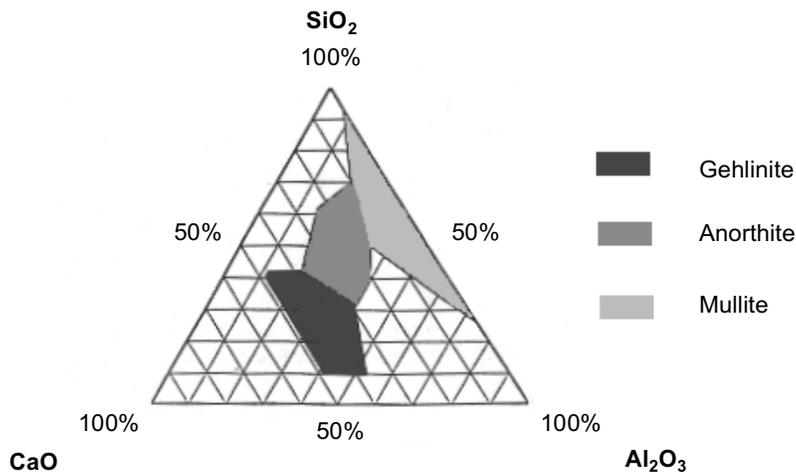


Figure 2.1 – Ternary Phase Diagram for Determination of Sulfate Resistance of Fly Ash (Ref 7)

The second mineral admixture evaluated in this test program is ground granulated blast furnace slag. Slag is primarily composed of calcium aluminosilicate glass, which reacts with the calcium hydroxide from the hydration of portland cement. This pozzolanic reaction, which is similar to the reaction described for low calcium fly ash, forms C-S-H and/or C-A-

H. The reaction reduces the available calcium hydroxide and thus reduces the potential for ettringite formation. Slag's effectiveness in improving sulfate resistance is highest at levels of replacement of 60 percent or higher as calcium hydroxide is more rapidly consumed at these levels (Ref 22).

2.4 Current Guidelines and Specifications for Sulfate Resistant Concrete

This section outlines the current guidelines and specifications available for engineers to use when designing a concrete for use in a sulfate environment. Guidelines provided by the American Concrete Institute (ACI) and the International Conference of Building Officials and specifications used by the Texas Department of Transportation are reviewed.

2.4.1 ACI and Uniform Building Code (UBC) Guidelines

ACI Committee 201, ACI Committee 318, and the International Conference of Building Officials have provided guidelines for categorizing the severity of different sulfate environments and for choosing proper materials and mix proportions for producing concretes for those environments. These guidelines have been referenced in several concrete mix design guides such as the Portland Cement Association's *Design and Control of Concrete Mixtures* (Ref 23) and ACI 211.1-91, "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete" (Ref 24). This section focuses on the guidelines for material selection and proportioning. Guidelines published in ACI 318-95, "Building Code Requirements for Structural Concrete and Commentary" (Ref 25); ACI 201.2R-92, "Guide to Durable Concrete" (Ref 8); and the 1994 edition of the *Uniform Building Code* (Ref 26) are summarized in Table 2.3.

The mild sulfate environment for these guidelines has sulfate concentrations so slight that sulfate attack is not a concern and guidelines for material selection are not necessary. For the moderate environments, the typical Type II cement is recommended for this level of sulfate resistance. However, the guidelines also acknowledge the benefits of mineral admixtures; Type IP(MS) and IS(MS) blended cements are recommended by ACI 201 and the UBC. ACI 318 designates three additional cements that can be used in moderate

environments where the Type P(MS), I(PM)(MS), and I(SM)(MS) blended cements are allowed. ACI 201 also allows the use of any combination of Type I cement with a mineral admixture that has been proven by tests to perform at or above the level of Type II cements. The mix proportions for a moderate environment are limited such that a water-to-cementitious material ratio of 0.5 or less is required in each guideline to ensure the use of good quality concrete. ACI 318 and the UBC also designate minimum compressive strength requirements for normal weight and lightweight aggregate concrete. For a moderate environment, the two guidelines require a minimum compressive strength of 27.6 MPa (4,000 psi).

Table 2.3 – Material and Proportion Recommendations for Normal and Lightweight Aggregate Concrete Subject to Sulfate Attack (Refs 8, 25, 26)

Severity of Environment	Cement Type	Maximum w/c by weight for Normal Weight Concrete	Minimum f'_c for Light and Normal Weight Concrete ⁵ (psi)
Mild	--	--	--
Moderate ¹	Type II, IP(MS), IS(MS), P(MS), I(PM)(MS), I(SM)(MS) ²	0.50	4,000
Severe	Type V ³	0.45	4,500
Very Severe	Type V + pozzolan or slag ⁴	0.45	4,500

1 When chloride or other depassivating agents are present in addition to sulfate, a lower water-cement ratio may be necessary to reduce corrosion of embedded items.

2 ACI 201 and the UBC designated Type II, IP(MS) and IS(MS) while ACI 318 recommended all six cement types shown in the table. ACI 201 also allows any blend of Type I cement and a ground granulated blast furnace slag or a pozzolan that has been determined by tests to give equivalent sulfate resistance.

3 ACI 201 also allows any blend of Type II cement and a ground granulated blast furnace slag or a pozzolan that has been determined by tests to give equivalent sulfate resistance.

4 Use a pozzolan or slag that has been determined by tests to improve sulfate resistance when used in concrete containing Type V cement.

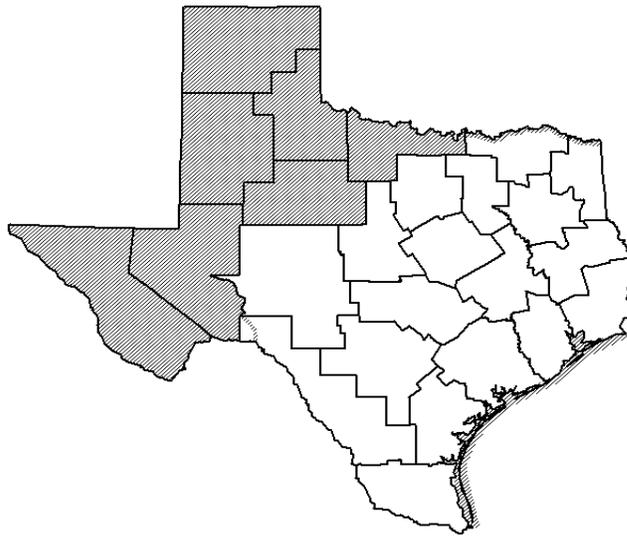
5 Minimum strength requirements only designated in ACI 318 and the UBC. Note that 1 psi = 0.006895 MPa.

For the severe environment, the high-sulfate resistant Type V cement is recommended in each guideline. ACI 201 acknowledges that a Type II cement improved to the performance level of a Type V cement by using mineral admixtures is also acceptable. The maximum water-to-cementitious material ratio in each guideline for a severe environment is 0.45. ACI 318 and the UBC require a minimum compressive strength of 31.0 MPa (4,500 psi) for normal and lightweight concrete. For the very severe environment, each guideline recommends a Type V cement augmented with a pozzolan or slag and a maximum water-to-cementitious material ratio of 0.45. ACI 318 and the UBC also recommend a minimum compressive strength of 31.0 MPa (4,500 psi).

2.4.2 *Current Practice of the Texas Department of Transportation*

The Texas Department of Transportation currently controls sulfate attack in the state of Texas by controlling the chemistry of the cementitious materials in areas of concern. TxDOT first established regions in the state where sulfates are known to be a concern. The areas of possible sulfate attack were determined by experience in each of the districts. The TxDOT *Bridge Design Guide* provides an illustration of the potential sulfate environments as shown in Figure 2.2 (Ref 27). The shaded areas on the map represent the areas where sulfate attack is a concern. The caveat to the original figure given below the map gives an idea of the accuracy and scope of the data it presents.

For the shaded areas of the map in Figure 2.2 — areas such as West Texas, the Panhandle, and the Gulf Coast — TxDOT requires the use of Type II portland cement. Type II cement is readily available in most of Texas and is commonly used in TxDOT projects. TxDOT also allows an optional secondary precaution of using fly ash with the Type II cement to prevent sulfate attack. The TxDOT specifications, however, acknowledge the negative effects of high calcium fly ashes on sulfate resistance. Item 421.2(2) of the TxDOT *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges* does not allow the use of the typically high calcium, ASTM Class C fly ashes when Type II cements are required (Ref 28). Texas specifications currently never require the use of Type V cements.



The shaded areas represent known areas of possible corrosion caused by sulphate soils or saltwater. There may be areas within the boundaries that are free from corrosive influence and areas outside the boundaries that are not. Careful consideration is recommended.

Figure 2.2 – “Figure 3.17c Protective Measures Against Corrosion from Below,” from TxDOT Bridge Design Guide (Ref 27)

2.5 Available Performance Tests for Evaluating Sulfate Resistance

The ultimate goal of this TxDOT project is to convert the types of specifications described in Section 2.4 into performance-based specifications so that the relevant concrete properties that affect sulfate attack are identified and performance tests are provided to evaluate these properties in concrete. Section 2.3 has provided an overview of the concrete properties affecting sulfate attack in concrete. This section will review the available tests that can be used to evaluate these properties. The tests that will be examined are ASTM C452-95, “Standard Test Method for Potential Expansion of Portland-Cement Mortars Exposed to Sulfate” (Ref 29); ASTM C1012-95, “Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution” (Ref 1); and USBR 4908, “Procedure for Length Change of Hardened Concrete Exposed to Alkali Sulfates” (Ref 4).

2.5.1 *ASTM C452-95*

The rapid mortar bar test, ASTM C452, “Standard Test Method for Potential Expansion of Portland-Cement Mortars Exposed to Sulfate,” was originally published and approved by ASTM C01.29, the Subcommittee for Sulfate Resistance, in 1960. The test method involves the measurement of expansion of mortar bars made from a combination of portland cement and gypsum. The gypsum in the mortar mix provides the source of sulfate that instigates rapid reactions in the specimens. The gypsum accelerates the development of and increases the amount of ettringite produced in the fresh and hardened concrete and thus accelerates the reactions typical of sulfate attack. The test method has been referred to in the ASTM standards for portland cement, ASTM C150, as an optional means for evaluating severe sulfate-resistant portland cements. ASTM C150 designates a maximum expansion limit of 0.04 percent at 14 days for Type V portland cements. ASTM Subcommittee C01.29 recommends limits of 0.06 percent expansion at 14 days for moderate sulfate-resistant Type II cements and 0.04 percent expansion at 14 days for severe sulfate-resistant Type V cements (Ref 30).

The major advantage of ASTM C452-95 is the short duration of the test. The sulfate resistance of a mortar can be evaluated in approximately 14 days. The major disadvantage of the test is that it has shown to be inaccurate when used for testing mortars made with blended hydraulic cements or blends of cement and a mineral admixture. The first problem is that the blended cements do not develop enough maturity in the 14-day measured expansion period. Secondly, the test does not represent field conditions because the gypsum incorporated into the mix exposes the mortar to sulfate attack in its fresh state before hydration has even occurred. This interferes with the natural hydration process. One reaction that the early sulfate exposure especially affects is the pozzolanic reaction that characterizes the benefit of mineral admixtures — the pozzolanic consumption shown earlier in Equation 2.6. There is no time for the portland cement hydration by-product, calcium hydroxide, to be absorbed by the mineral admixture’s pozzolanic constituents before the paste is exposed to sulfate attack (Ref 30). The advantages of reduced potential for ettringite formation and lowered permeability are minimized. These flaws in the test have led researchers to limit the scope of

ASTM C452. The test should only be used to test plain portland cement mortars because the test does not accurately identify the benefits of mineral admixtures.

2.5.2 *ASTM C1012-95*

With knowledge of the limits of ASTM C452, ASTM Subcommittee C01.29 began researching the development of a new performance test that would be applicable to portland cement, blended hydraulic cements, and blends of portland cement with pozzolans and/or slag. The subcommittee, led by chairman Katherine Mather, initiated two cooperative test programs at eleven different laboratories to test several types of portland and blended cements (Refs 30, 31). The result of this work was the formation and standardization of the mortar bar test, ASTM C1012, in 1984. For the new test, the method of adding sulfate into the mortar during mixing is eliminated. Instead, sulfate exposure is provided by immersing the mortar bars into a sulfate solution after the mortar has reached a certain strength. Beginning sulfate exposure when mortars are at an equivalent strength value is said to simulate actual concrete practice in that concrete in the field will typically be at approximately the same strength when sulfate attack begins regardless of the cementitious chemistry (Ref 1). The type of solution used for the test and the strength requirement has varied over the years, but the subcommittee has currently settled on using a 0.352 molar sodium sulfate (Na_2SO_4) solution and a strength requirement of 19.7 MPa (2,850 psi) before immersion.

Through analysis of test results and correlation with ASTM C452 limits, ASTM C01.29 was able to establish expansion criteria to correspond with the ASTM C1012 test. The test criterion requires a maximum expansion limit of 0.10 percent at 180 days of sulfate solution exposure for moderate sulfate resistance and a limit of 0.05 percent at 180 days for severe sulfate resistance (Refs 30, 31). These expansion limits were supported by the two original test programs and by a third program performed by twelve laboratories in January of 1988 (Ref 30). Ouyang computed expansion limits for ASTM C1012 by correlating limits with peak expected compressive strengths in the bars, and the calculated values agreed well with the values established by ASTM C01.29 (Ref 32). In all the testing, the test and its

corresponding criteria were shown to accurately evaluate the performance of mortars containing portland cement, blended cements, and blends of cement with mineral admixtures. Unlike ASTM C452, results correlated well with field and laboratory experience. The only disadvantage of ASTM C1012 is that the test may be considered to be too slow to be an accelerated test as it requires 6 months before significant results are obtained. The clear advantage of the test is that it provides a reliable way for engineers to evaluate the sulfate resistance of all types of cementitious material combinations.

2.5.3 *USBR 4908*

Both tests described thus far evaluate the resistance of mortar to sulfate attack and not the actual concrete. For engineers interested in better representing service conditions by testing actual concrete specimens, the United States Bureau of Reclamation (USBR) has provided the standardized test, USBR 4908, “Procedure for Length Change of Hardened Concrete Exposed to Alkali Sulfates” (Ref 4). The USBR 4908 procedures are similar to ASTM C1012 in that test specimens are soaked in a sulfate solution after the specimens have reached a certain strength, but the specimens in this test are concrete cylinders instead of mortar bars. The expansions of the cylinders are measured as well as the mass change during immersion. USBR 4908 is applicable to concrete mixes containing portland cements, blended cements, or blends of cement and mineral admixtures.

The test procedures provide three methods in which the type of soaking is varied for each method (Ref 12). Method A involves continuous soaking of the cylinders in a 2.1 percent sodium sulfate (Na_2SO_4) solution; Method B involves continuous soaking in a 10 percent sodium sulfate solution; and Method C is a soaking/drying test where the cylinders are alternately soaked for 16 hours in a 2.1 percent solution and then dried for 8 hours under a forced air draft of $54\text{ }^\circ\text{C}$ ($130\text{ }^\circ\text{F}$). Method B and the drying/soaking Method C are equally rigorous tests while Method A requires more time to show deterioration. Method B has been found to be a true accelerated test with no apparent irregularities in the mechanisms of sulfate attack (Ref 12). Even for the more rigorous methods, the USBR 4908 test requires at least 1 to 2 years before any significant results can be obtained. There are currently no widely

accepted expansion limits or mass change limits that go along with these procedures. The main advantage of the USBR 4908 test is its flexibility. It can be used to evaluate the effects of permeability, mineral and chemical admixtures, and other mix design alternatives as well as various curing procedures on the sulfate resistance of concrete.

2.6 Proposed Performance-Based Specifications for Sulfate Resistance

The state-of-the-art literature review performed earlier in this project was followed up by a proposal for a new form of specifications for the Texas Department of Transportation (Ref 6). The guidelines are outlined in Tables 2.4 and Table 2.5. The proposed performance-based specifications provide a means for evaluating the severity of an environment where concrete will be in service as well as a means for evaluating the properties of a concrete to ensure the durability of the concrete in its environment. The focus of this section will be on the specifications developed for evaluating concrete properties for application in a sulfate environment.

Table 2.4 – Mix Design Parameters for Concrete Resistant to Sulfate Attack (Ref 6)

Severity of Environment	Cement Type	Maximum w/c ¹
Mild	--	--
Moderate ²	Type II ³	0.50
Severe	Type V ⁴	0.45
Very Severe	Type V + pozzolan or slag ⁵	0.45

1 A higher water-cement ratio may be permitted if the performance requirements of Table 2.5 are met or exceeded.

2 When chloride or other depassivating agents are present in addition to sulfate, a lower water-cement ratio may be necessary to reduce corrosion of embedded items.

3 Or use a combination of cement and mineral admixture that meets or exceeds the performance of Type II cement, as tested by ASTM C 1012-95, and the performance requirements of Table 2.5.

4 Or use a combination of cement and mineral admixture that meets or exceeds the performance of Type V cement, as tested by ASTM C 1012-95, and the performance requirements of Table 2.5.

5 Use a pozzolan or slag that has been determined by tests to improve sulfate resistance when used in concrete containing Type V cement.

Table 2.5 – Performance Criteria for Sulfate Resistance by Modified ASTM C1012-95 (Ref 6)

Performance Criteria	Resistance Level	
	Moderate	High
Expansion, %	0.10	0.05
Mass Loss, %	0.00	0.00

The proposed specifications are similar to the ACI and UBC guidelines described earlier. ACI Committee 201 was close to the idea of performance-based specifications in their guidelines. The recommendations provided an option for the engineer where any cementitious material combination could be used as long as it was tested and proven to work as well as or better than the recommended portland cement type. The problem in the guideline was that no particular test was specified for doing these evaluations. The proposed specifications in Tables 2.4 and 2.5 provide the next step because a performance test has been selected for evaluating mortars. ASTM C1012-95 was selected because of its applicability to all types of cements and cement-mineral admixture combinations and the availability of well supported performance criteria corresponding to the test. The expansion limits for ASTM C1012-95 discussed in Section 2.5 were incorporated into the performance-based specifications as shown in Table 2.5.

In developing these specifications, the idea of a mass loss criteria corresponding to ASTM C1012-95 was also proposed. As shown in Table 2.5, it would be required that the mortar bar test specimens experience no loss of mass after 180 days of soaking in order to meet moderate or severe sulfate resistance levels. The basis of this proposal came from Freeman’s research discussed earlier where it was shown that some concrete with low permeability may not suffer damage from expansion; however, the concrete may suffer from surface scaling and thus experience mass loss.

While the idea of incorporating a device for identifying sulfate damage from surface scaling in performance evaluations is good, further investigation and study of the procedures of ASTM C1012-95 indicates this particular proposal may not work. One major problem was the fact that surface scaling is a typical damage mechanism in concrete that may not occur in mortars. Surface scaling occurs when the mortar along the surface of a concrete

breaks away from the coarse aggregate as the mortar expands or deteriorates. Freeman was testing concrete specimens using the USBR 4908 procedures and observed this type of damage. In the ASTM C1012 test, it is unlikely that the mortar bars will display this same type of damage because there is no coarse aggregate from which the surface mortar can break away. A second problem is that the mass loss criteria are generally for identifying problems with the resistance of low permeability mortars to sulfate attack. In general, ASTM C1012 cannot be used to test low permeability mortars because mortar proportions are set. Changing the mortar proportions would make the expansion limits inapplicable. A final problem with the mass loss criteria is the difficulties a mass loss measurement would present to the ASTM C1012 procedures. Currently in the ASTM C1012-95 procedures, mortar bar specimens are kept fully saturated throughout curing, soaking in sulfate solution, and measuring length change. A mass measurement could not be made for the bar in its saturated state because the bar would need to be dried to a surface-saturated dry (SSD) condition. This drying would be undesirable as the bars are extremely sensitive to moisture change, and the drying may cause shrinkage and interfere with the bar expansion. Alternative suggestions for incorporating mass loss evaluations into specifications will be presented during the final conclusions and recommendations of the project.

In addition to evaluations of the performance of the cementitious materials of the concrete, these specifications provide requirements for the water-to-cementitious material ratios of the concrete to be used in each environment. These water-to-cementitious material ratio limits were taken directly from the ACI and UBC recommendations. The goal of the limits is to ensure that a moderate or low permeability concrete is used in sulfate environments. There is a concern with these water-to-cementitious material ratio limits as questions arise as to where the values came from and why they are necessary. Another concern is that the values do not address the fact that low permeability can be reached in several ways. With the emergence of pozzolans and slag over the past few decades, high water-to-cementitious material ratios no longer necessarily mean high permeability concrete. A more appropriate performance-based criterion may be the specified maximum permeability values that the concrete must meet.

The next goal in this long-term research project for the Texas Department of Transportation is to refine and improve the specification proposal and verify that the guidelines work through an experimental investigation. The proposed specifications and their corresponding problems discussed in this section have led to the formation of this test program. The first phase of the test program involves evaluating the accuracy of ASTM C1012-95 and its expansion criteria. The second phase involves answering questions about the impact of low permeability and determining a proper way to specify permeability for concrete that is durable against sulfate attack. Two questions that will be addressed are whether the water-to-cementitious material ratio is always directly related to permeability and whether it is the water-to-cementitious material ratio or permeability itself that influences the sulfate resistance of concrete. The ultimate goal is the finalization of these specifications into something TxDOT can rely on as a guideline for efficiently producing concrete resistant to sulfate attack.

Chapter Three: Experimental Program

3.1 Introduction

The testing program reported herein involved a two-phase investigation of sulfate attack in concrete. The first phase of the project, Phase 1, involves the evaluation of the mortar bar performance test, ASTM C1012-95. Mortars with varying cementitious materials were tested to examine the accuracy and consistency of the test. Phase 1 testing began in the summer of 1998 and ended in the spring of 1999. The second phase, Phase 2, involved the examination of the effect of permeability on the sulfate resistance of concrete using the USBR 4908 Method B testing procedure. Non-air-entrained concretes with varying cementitious materials and water-to-cementitious material ratios were cured under different conditions and then tested to see how these varying properties affected the permeability and the sulfate resistance of the concrete. Testing for Phase 2 began in the fall of 1998. Long-term results for this testing will not be obtained until the spring of 2001. All testing for this project was performed at the Construction Materials Research Group (CMRG) Laboratory in the J.J. Pickle Research Campus of The University of Texas at Austin.

3.2 Phase 1 Testing Program

Phase 1 involved the testing of twenty-four different mortars according to ASTM C1012-95. ASTM C1012-95 consists of measuring the expansion of mortar bars that have been soaking in a sodium sulfate solution. The expansion after 180 days of soaking is the critical measurement because this value is used as a criterion for determining sulfate resistance.

Plain portland cement mortars, mortars with cement combined with fly ash, and mortars with cement combined with slag were tested using ASTM C1012-95. Four cements, two types of fly ash, and one ground granulated blast furnace slag (GGBFS) were used. One ASTM C150 Type I, two Type I-II, and a Type V portland cement were tested. Six material combinations were evaluated for each cement. The cementitious material combinations considered for each cement are as follows:

1. Plain portland cement,
2. Portland cement combined with an ASTM Class F fly ash at 20 percent and 30 percent volumetric replacement,
3. Portland cement combined with an ASTM Class C fly ash at 25 percent and 35 percent volumetric replacement, and
4. Portland cement combined with a slag at 50 percent volumetric replacement.

3.2.1 *Materials*

The following sections provide information on the materials used in the mortar mixes for this phase. The portland cements, fine aggregate, and mineral admixtures are described. Potable water was used for all mixing.

Portland Cement

Four commercially available portland cements in the state of Texas were evaluated in this testing program: one Type I, two Type I-II, and a Type V cement. The Type I cement is from Buda, the two Type I-II cements are from Midlothian and New Braunfels, and the Type V cement came from Odessa. Chemical analyses for the four cements are provided in Table 3.1. The data provided in this table came from mill certificates obtained from the cement suppliers. A specific gravity of 3.15 was assumed for proportioning for all the cements. As denoted in the table, the Type I-II cement from Midlothian will be referred to as the Type I-II (A) cement in this report, while the Type I-II cement from New Braunfels will be referred to as the Type I-II (B) cement.

The most important chemical property reported in Table 3.1 is the tricalcium aluminate (C_3A) content of the cements. The Type I cement has no ASTM C150 limit for C_3A content, thus the high 12 percent value is acceptable. The Type I-II (A) cement has a C_3A content of 5.1 percent. This value is considerably lower than the ASTM C150 maximum limit of 8 percent for Type II cement and is just above the 5 percent limit required

for Type V cements. The 7 percent C₃A content of the Type I-II (B) cement is just below the ASTM C150 limit of 8 percent for Type II cements. Finally, the Type V cement meets the C₃A content limit of 5 percent for sulfate resistant cement because the cement contains zero C₃A.

Table 3.1 – Chemical Properties of Portland Cements

Cement Type (ASTM C150) Source	Portland Cements			
	I Buda, TX	I-II (A) Midlothian, TX	I-II (B) New Braunfels, TX	V Odessa, TX
Silicon Dioxide (SiO ₂), %	--	20.42	20.9	21.86
Aluminum Oxide (Al ₂ O ₃), %	--	4.42	4.5	3.18
Ferric Oxide (Fe ₂ O ₃), %	--	3.94	3.2	5.66
Magnesium Oxide (MgO), %	1.3	1.06	1.4	0.75
Sulfur Trioxide (SO ₃), %	3.5	2.96	3.0	3.06
Equivalent Alkalies, %	0.64	--	0.43	0.38
Loss on Ignition (LOI), %	1.8	0.76	1.3	0.67
Insoluble Residue (IR), %	0.16	0.13	0.15	--
Dicalcium Silicate (C ₂ S), %	--	11.70	--	21.8
Tricalcium Silicate (C ₃ S), %	--	62.10	61	54.2
Tricalcium Aluminate (C ₃ A), %	12	5.10	7	0.0

Fine Aggregate

The fine aggregate used for making the mortars was a graded Ottawa sand meeting the requirements of ASTM C778-97 (Ref 33). ASTM C1012-95 requires that mortars for this test be made with this well-graded, rounded, particle sand from Ottawa, Illinois. The sand is predominately graded between the 600 µm (No. 30) and 150 µm (No. 100) standard sieve sizes such that a very fine, well-distributed aggregate is created. The sand has a specific gravity of 2.65 and an absorption capacity of 0.5 percent.

Mineral Admixtures

Three different types of mineral admixtures were used in mixes for Phase 1 testing. A low-calcium ASTM Class F fly ash; a high-calcium ASTM Class C fly ash; and a ground

granulated blast furnace slag were used. The ASTM Class F fly ash came from Rockdale, Texas; the ASTM Class C fly ash came from Parrish, Texas; and the slag was obtained from New Orleans, Louisiana. The chemical and physical properties of the mineral admixtures reported by the suppliers are shown in Table 3.2. The relevant chemical components for the fly ashes regarding sulfate attack of concrete are the calcium oxide (CaO), silicon dioxide (SiO₂), aluminum oxide (Al₂O₃), and iron oxide (Fe₂O₃) components. The Dunstan and Tikalsky-Carrasquillo methods for determining the effectiveness of a fly ash in improving sulfate resistance refer to these chemical components in their analysis.

Table 3.2 – Chemical and Physical Properties of Mineral Admixtures

Mineral Admixture	ASTM Class F Fly Ash	ASTM Class C Fly Ash	GGBF Slag
Source	Rockdale, TX	Parrish, TX	New Orleans, LA
Silicon Dioxide (SiO ₂), %	47.8	30.59	--
Aluminum Oxide (Al ₂ O ₃), %	22.6	17.78	--
Iron Oxide (Fe ₂ O ₃), %	5.9	5.85	--
SiO ₂ + Al ₂ O ₃ + Fe ₂ O ₃ , %	76.3	54.22	--
Calcium Oxide (CaO), %	17.3	27.55	--
Magnesium Oxide (MgO), %	2.3	4.65	--
Sulfur Trioxide (SO ₃), %	0.7	2.86	3.51
Sulfide Sulfur (S), %	--	--	1.3
Loss on Ignition, %	0.1	0.07	--
Moisture Content, %	0.1	0.12	--
% Retained on #325 Sieve	20.4	12.12	0.30
Specific Gravity	2.53	2.65	2.86

3.2.2 Mix Proportioning

The mix proportioning for the mortars is as required by ASTM C1012-95. Proportioning consisted of adding one part cement to 2.75 parts graded standard sand by weight. For the plain portland cement mortars, the procedures require a water-to-cement ratio of 0.485 for non-air-entrained cements and 0.460 for air-entrained cements. Because all the cements used in this test program are non-air-entrained cements, a water-to-cement ratio of 0.485 was used for these mixes. The basic mix proportions for the plain portland cement

mortars are shown in Table 3.3. Details for all the mixes performed for Phase 1 testing are provided in Appendix A. The air content for the mortars was assumed to be 2 percent. The water content is based on the assumption that the sand is completely dry at the time of mixing and the sand has an absorption of 0.5 percent. All batches for Phase 1 testing were approximately 0.006 cubic meters (0.2 cubic feet).

Table 3.3 – Mix Proportions for Plain Portland Cement Mortars

Material	Content (lb/yd ³)
Water	447
Portland Cement	897
Graded Sand	2,455
Conversion: 1 lb/yd ³ = 0.5933 kg/m ³	

When mineral admixtures were used, a specified volumetric percent of the cement was replaced by an equivalent volume of the mineral admixture. The Class F fly ash was used to replace 20 and 30 percent of the cement; the Class C fly ash was used to replace 25 and 35 percent of the cement; and the slag was used to replace 50 percent of the cement. For mortars with these blends of portland cement with fly ash or slag, the required water-to-cementitious material ratio was designated in two manners. Both requirements aimed at ensuring that the flow and consistency of the mortars with admixtures was similar to the plain portland cement mortars with which they were being compared. The flow of a mortar is quantitatively determined using a flow table test that produces a flow number. The test setup is described in ASTM C230-97 (Ref 34), and the procedures are described in ASTM C109-95 (Ref 35). For determining water contents for mixes containing a blend of cement with a pozzolan or slag, ASTM C1012-95 states the water-to-cementitious material ratio shall develop a flow within ± 5 of the flow number found for the plain portland cement mortar with a 0.485 water-to-cement ratio. Also, ASTM C1012-95 refers to the ASTM C109-95 procedures where it is recommended that the water-to-cementitious material ratio produce a flow number within the range of 110 ± 5 . For this testing program, the goal was to meet both

these requirements when possible to ensure the mortars with admixtures compared well with their plain portland cement mortar control as well as with all the other mortars made during the test program. Trial batches were performed for each cement-mineral admixture combination to determine the water content needed to produce the desired flow for each mix.

3.2.3 *Mixing, Placement, and Curing Procedures*

Mortars were mixed following the procedures of ASTM C305-94 (Ref 36). A two-speed electrically driven mechanical mortar mixer was used. The mixtures were made using the following procedures:

1. Place all the mixing water in the bowl.
2. Add the cement and mineral admixtures to the water, and then start the mixer at the slow speed. Mix for 30 seconds.
3. Add the sand slowly over a 30-second period while the mixer runs at the slow speed.
4. Stop the mixer and change it to the medium speed. Run the mixer for 30 seconds at this speed.
5. Stop the mixer and let the mortar stand for 1.5 minutes. Quickly scrape the mortar from the sides of the bowl and the mixing paddle back into the batch for the first 15 seconds. For the remainder of the rest time, cover the mixing bowl.
6. Finish by mixing for 1 minute at medium speed.

At the conclusion of mixing, a flow table test was run following the procedures described in Section 10.3 of ASTM C109-95. A flow number describing the consistency of the mix was obtained. After running the flow test, the tested mortar was returned to the mixer, and the mixer was run for another 15 seconds at medium speed to reincorporate the mortar.

The specimens required for ASTM C1012-95 are 25-mm x 25-mm x 286-mm (1-in. x 1-in. x 11.25-in.) mortar bars for expansion measurements and 51-mm (2-in.) cubes for

compressive strength tests. Six mortar bars were made along with eighteen cubes from each batch. Steel molds meeting the requirements of ASTM C490-97 (Ref 37) were used for casting the mortar bars. The molds are designed such that 19-mm (3/4-in.) stainless steel gauge studs can be embedded in each end of the bars. The studs are aligned such that a gauge length of approximately 254 mm (10 in.) between gauge stud ends exists. Brass cube molds meeting the requirements of ASTM C109-95 were used for the cube specimens. All molds were prepared for mixing by placing a light mold oil along the interior surfaces of the mold. This oil serves as a release agent for demolding. A bead of grease was run between the base plates of the molds and the frame to ensure the bottom of the mold was watertight. The mortar bars were placed following the procedures of ASTM C157-93 (Ref 38). The mortar for the cubes was placed following the procedures described in ASTM C109-95.

ASTM C1012-95 designates the ASTM C684-95 (Ref 39) Procedure A (Warm-Water Method) for initial curing of the mortar specimens after placement. The procedure involves sealing the top of the molds with a rigid steel, glass, or plastic plate such that the mold is completely watertight. The molds are then immersed in a curing tank of water that is kept at $35 \pm 3 \text{ }^\circ\text{C}$ ($95 \pm 5 \text{ }^\circ\text{F}$). The molds are kept in the tank for 23.5 ± 0.5 hours. For this test program, it was decided to modify this procedure so that the specimens were stored in a 100 percent relative humidity environment instead of being submerged in a tank. Attempting to create a completely watertight mold proved to be time consuming and risky because a leak in the mold would result in loss of the specimens. The modified procedure involved placing the mortar specimens in an environmental chamber that was kept at $35 \pm 3 \text{ }^\circ\text{C}$ ($95 \pm 5 \text{ }^\circ\text{F}$). The molds were covered with moist cloths and then sealed in plastic bags. Water trays were also placed inside the chamber to keep the humidity high. All these steps were done to ensure a 100 percent relative humidity environment was produced so that the fresh mortars did not lose any water. Creating this environment serves the same purpose as immersing the specimens in a curing tank. The molds were kept in the chamber for 23.5 ± 0.5 hours.

After the initial accelerated curing period in the environmental chamber, the molds were removed from the chamber, and the specimens were demolded. After demolding, most of the specimens were transferred to a curing tank containing saturated limewater kept at 23

± 1 °C (73.4 ± 3 °F). Two of the cubes were taken and tested for determining the compressive strength at 1 day. Compressive strength was measured following the procedures of ASTM C109-95. The cubes were cooled to ambient temperature in a moist cloth and then tested. ASTM C1012-95 requires that the mortar specimens are cured until the mortar displays a strength of 19.7 MPa (2,850 psi) or higher. Cubes were tested until the average strength of the cubes was greater than 19.7 MPa (2,850 psi). Until this strength was reached, all the specimens were kept in the saturated limewater.

3.2.4 *Testing Procedures*

Upon reaching the strength of 19.7 MPa (2,850 psi), the mortar bar specimens were measured for their initial length. Length measurements were made using a length comparator following the procedures of ASTM C490-93. A 295.275-mm (11.625-in.) reference bar made of a low coefficient of thermal expansion steel alloy was used to zero the gauge on the comparator. The mortar bars were kept in a saturated state during measuring because any drying could result in shrinkage. Before taking measurements, the studs at the bar ends were blotted dry. The bars were then placed in the comparator. The bars were spun a few times, and the maximum reading from the dial gauge was taken. Length measurements were recorded to the nearest 0.001 mm (0.0001 in.). Between each measurement, the reference bar was placed back in the comparator to ensure the gauge remained zeroed.

After initial length measurements, the mortar bars were immediately immersed in a sodium sulfate solution. The bars were stored in containers with lids so that the solution would not evaporate. The bars were supported by several 13-mm (1/2 in.) diameter PVC pipes so that no surface of the bars touched any side of the container. Following test requirements, the sodium sulfate solution was prepared at least 1 day before its use. The solution designated by ASTM C1012-95 was a 0.352 molar, 5 percent sodium sulfate (Na_2SO_4) solution. One liter of solution was prepared by first dissolving 50.0 g of anhydrous technical grade Na_2SO_4 salt in 900 mm of deionized water. After the salt was dissolved, the solution was diluted to 1 liter by adding deionized water. The solution was required to have a pH between 6 and 8. Throughout all the testing, the sodium sulfate solutions were kept at

$23 \pm 1^{\circ}\text{C}$ ($73.4 \pm 3^{\circ}\text{F}$). ASTM C1012-95 requires that the mortar bars be stored such that a sulfate solution to bar volume ratio of 4 ± 0.5 exists. Solutions were made for this test program so that a ratio of 4.07 was maintained.

Length measurements were taken for each batch at 1, 2, 3, 4, 8, 13, and 15 weeks after the bars were initially placed in the sodium sulfate solution. Long-term measurements were taken after 4 and 6 months of soaking. Months were considered to be 30 days. The length measurement procedures described earlier were followed for all measurements. After each measurement, the sulfate solution for the bars was replaced with fresh solution, and the containers and pipe supports were cleaned.

The data obtained from this testing were the change in length of the mortar bars over time in the sodium sulfate solution. Length measurements were used to calculate percent expansions for the bars. The visual appearance of the specimens was also observed, and any cracking or warping in the bars was documented.

3.3 Phase 2 Testing Program

The second phase of the testing program involved the investigation of the influence permeability has on the sulfate resistance of concrete. Plain portland cement concrete mixes made using three different types of cement were evaluated. Concretes with combinations of portland cement and two types of fly ash were also tested. For each cement and combination of cement with fly ash, mixes with at least three different water-to-cementitious material ratios were tested. The cementitious content of all the mixes was kept the same so that the chemistry in the concrete remained constant while the permeability varied. Variation in permeability was further achieved by curing specimens from each batch with different methods. Some specimens were cured at room temperature while others were cured using accelerated curing procedures. The permeability for all the concretes was determined using ASTM C1202-94. Sulfate resistance was evaluated by making and testing concrete cylinders following the procedures of USBR 4908 Method B. Details on the materials used in testing, the mix designs that were tested, and the testing procedures are discussed in the following sections.

3.3.1 Materials

This section describes the materials used for the concrete batches prepared for Phase 2 testing. All the portland cement and mineral admixtures used in the Phase 2 mixes were the same materials that were tested in Phase 1. The Type I cement from Buda, Texas; the Type I-II (B) cement from New Braunfels, Texas; and the Type V cement from Odessa, Texas were used in the Phase 2 mixes. For mineral admixtures, the ASTM Class F fly ash from Rockdale, Texas and the ASTM Class C fly ash from Parrish, Texas were used. Potable water was used in all the mixes. The remaining sections describe the aggregates and the chemical admixture used in testing.

Fine Aggregate

The fine aggregate used in Phase 2 testing was a commercially available natural siliceous concrete river sand from Austin, Texas. Sieve analyses of the sand indicated it met the requirements of ASTM C33-97 (Ref 40) for fine aggregate. An example of a sieve analysis is shown in Table 3.4. The sand had an average fineness modulus of 2.60, a dry bulk specific gravity of 2.61, and an absorption capacity of 0.8 percent.

Table 3.4 – Sieve Analysis of Phase 2 Fine Aggregate

Sieve Size	% Passing	ASTM C33-97 Spec. (Ref 40)
9.5-mm (3/8-in.)	100	100
#4	100	95-100
#8	93	80-100
#16	73	50-85
#30	49	25-60
#50	20	10-30
#100	2	2-10
#200	0	0-5

Coarse Aggregate

The coarse aggregate used in Phase 2 testing was a commercially available natural concrete river gravel from Austin, Texas. According to sieve analysis, the coarse aggregate met the requirements for a No. 67 coarse aggregate designated by ASTM C33-97. Particle sizes generally ranged from 19 mm (3/4 in.) to a No. 4 sieve. A sieve analysis from the supplier and an analysis performed at the CMRG laboratory are shown in Table 3.5, and results are compared with the ASTM C33-97 specifications. While there were slight differences in particle gradation between the reported and laboratory analyses, both met ASTM Size 67 requirements. Analysis also showed the coarse aggregate had an average dry-rodded unit weight of 61 kg/m³ (103 lbs/ft³) and a dry bulk specific gravity of 2.58. The absorption capacity of the aggregate was determined to be about 1.0 percent.

Table 3.5 – Sieve Analysis of Phase 2 Coarse Aggregate

Sieve	Size	% Passing (Supplier)	% Passing (Laboratory)	ASTM C33-97 Spec. (Ref 40) for Size #67
	25-mm (1-in.)	100	100	100
	19-mm (3/4-in.)	92	95	90-100
	16-mm (5/8-in.)	78	--	--
	13-mm (1/2-in.)	53	66	--
	9.5-mm (3/8-in.)	28	48	20-55
	#4	8	7	0-10
	#8	2	1	0-5

Chemical Admixture

A high range water-reducing (HRWR) admixture was used for some of the mixes in this test program in order to achieve workability in a mix without increasing the water content. The HRWR admixture, also referred to as a *superplasticizer*, was especially useful for the low water-to-cementitious material ratio mixes because these batches had extremely low water contents and thus little workability upon initial mixing. The HRWR admixture used for this project was a commercially available product from Cleveland, Ohio. The

admixture meets the requirements of ASTM C494-92 (Ref 41) for Type A and F water-reducing admixtures.

3.3.2 *Mix Designs*

A total of twenty-two concrete mix designs were evaluated in Phase 2. Ten plain portland cement concretes and twelve concretes containing a combination of cement with fly ash were evaluated.

For the plain portland cement mixes, three types of cement were tested with at least three different water-to-cement ratios. The batches are outlined below:

1. Concrete with Type I cement and water-to-cement ratios of 0.35, 0.45, and 0.55.
2. Concrete with Type I-II (B) cement and water-to-cement ratios of 0.35, 0.45, 0.50, and 0.55.
3. Concrete with Type V cement and water-to-cement ratios of 0.35, 0.45, and 0.55.

For the concrete mixes containing fly ash, only the Type I-II (B) cement was used for testing. The following batches were considered:

1. Concrete with ASTM Class F fly ash at 20 percent and 30 percent volumetric replacement with water-to-cementitious material ratios of 0.35, 0.45, and 0.55.
2. Concrete with ASTM Class C fly ash at 25 percent and 35 percent volumetric replacement with water-to-cementitious material ratios of 0.35, 0.45, and 0.55.

The general mix designs used for the plain portland cement concrete mixes are shown in Table 3.6. Appendix B outlines the mix proportions for all the mixes done in Phase 2 testing.

Table 3.6 – Mix Proportions for Plain Portland Cement Concretes

Water-to-Cement Ratio	0.35	0.45	0.50	0.55
Water (lb/yd ³)	197	254	282	310
Cement (lb/yd ³)	564	564	564	564
Coarse Aggregate – dry (lb/yd ³)	1,780	1,780	1,780	1,780
Fine Aggregate – dry (lb/yd ³)	1,419	1,362	1,334	1,306
Superplasticizer (fl oz/100 lb cement)	10-18	0-8	0	0
Desired Slump (in.)	0.5-2	3-5	5-7	6-8
Conversions: 1 lb/yd ³ = 0.5933 kg/m ³ and 1 in. = 25.4 mm				

The water content and aggregate weights shown in Table 3.6 do not include adjustments made for aggregate moisture content. Ranges of HRWR admixture quantities are given for each mix. The amount used varied depending on the water-to-cement ratio, cement type, and the mixing conditions. The amount of the chemical admixture that was expected to be used was taken out of the water content before mixing because the superplasticizer is mostly water. The superplasticizer was generally used for the 0.35 and 0.45 water-to-cement ratio mixes to produce workable concrete.

The original goal was to use the superplasticizer to make all the mixes have a slump value in the range of 102 to 204 mm (4 to 8 in.). This proved to be a difficult task for the 0.35 water-to-cement ratio mixes because of the medium level cement content and low water-to-cement ratio, which led to a low water content and an extremely dry mix. It was found early in testing that pushing these mixes to a 102-mm (4-in.) or higher slump created a poor quality concrete. The fresh concrete showed high levels of bleeding, and the hardened concrete had a black appearance. Long-term testing showed the mixes' 28-day strengths were as low as the equivalent 0.55 water-to-cement ratio mixes and their permeability was not as low as expected. A decision was made to reduce the slump requirement for the 0.35 water-to-cement ratio mixes to 13 to 51 mm (0.5 to 2 in.). These lower slump concretes performed well as long as care was taken to properly consolidate the material during placement.

For the concrete mixes with combinations of cement with fly ash, a certain volumetric percentage of the cement content was replaced by the fly ash. Because the fly ashes have lower specific gravity values than the cements, this replacement generally resulted in a drop in the total weight of cementitious material. To keep the water-to-cementitious material ratios at 0.35, 0.45, and 0.55, water contents had to be dropped slightly. The quantity of fine aggregate in the mix had to be increased to keep the yield of the mix the same. Mix designs for blends of cement and fly ash were done for volumetric replacement levels of 20, 25, 30, and 35 percent. Table 3.7 shows the mix proportion adjustments used when the ASTM Class F fly ash was used at a replacement level of 20 percent. Appendix B details each mix with all the different fly ash replacement levels.

Table 3.7 – Mix Proportions for Concrete Containing Blend of Portland Cement and Fly Ash (20 percent volumetric replacement)

W/C Ratio	0.35	0.45	0.55
Water (lb/yd ³)	190	244	298
Portland Cement (lb/yd ³)	451	451	451
Fly Ash (lb/yd ³)	91	91	91
Coarse Aggregate – dry (lb/yd ³)	1,780	1,780	1,780
Fine Aggregate – dry (lb/yd ³)	1,449	1,395	1,340
Superplasticizer (fl oz/100 lb cement)	10-18	0-8	0
Desired Slump (in.)	0.5-2	3-5	6-8
Conversions: 1 lb/yd ³ = 0.5933 kg/m ³ and 1 in. = 25.4 mm			

3.3.3 *Mixing, Placement, and Curing Procedures*

The concrete batches were mixed using a 0.11 cubic meter (4-cubic foot) rotary drum mixer meeting the requirements of ASTM C192-95 (Ref 42). Batch sizes were typically 0.06 cubic meters (2 cu ft). Basic mixing procedures followed the guidelines of ASTM C192-95. Before batching out materials, the moisture contents of the coarse and fine aggregates were measured, and moisture adjustments were made for the batch weights. Before placing any

materials in the mixer, the sides of the drum were buttered with a mortar approximately equal to 10 percent of the batch mortar to compensate for mortar loss because of sticking to the sides of the mixer. Initial mixing was done following these steps:

1. Add coarse aggregate and some of the mixing water before starting the mixer.
2. Start the mixer and then add the fine aggregate, cement, and any mineral admixtures used. Finish by adding the remainder of the mixing water.
3. Mix the ingredients for 3 minutes. Then stop the mixer for a 3-minute rest period. Scrape any mortar from the drum sides and blades at the beginning of the rest time. Cover the mixer for the remainder of the time. Finish by mixing for another 2 minutes.

After this initial mixing, the concrete was generally tested for its fresh properties. The slump of the concrete was determined following the procedures of ASTM C143-90 (Ref 43). Air content was only determined for trial batches to ensure the assumed air content was fairly accurate. The pressure method described in ASTM C231-97 (Ref 44) was used to determine the air content. For batches where the slump was found to be acceptable after this initial mixing, the concrete tested for slump was reincorporated into the mix by mixing for 15 to 30 seconds, and the concrete was ready for placement. For the low and moderate water-to-cementitious material ratio mixes, the slump of the concrete was generally very low after initial mixing. For these low slump mixes, additional mixing was needed to incorporate the superplasticizer into the mix to achieve more workability. The HRWR admixture was added to the ingredients using a syringe while the mixer was running. When small dosages of the admixture were used, the mixer was run for approximately 5 minutes. Larger dosages required more time for incorporation, and the mixer was run for 8 to 10 minutes. After incorporating the superplasticizer, the fresh concrete was tested for slump. The process was repeated until the desired slump was reached and the concrete was ready for placement.

The specimens needed for this testing program were 76-mm (3-in.) diameter by 152-mm (6-in.) long cylinders for early compressive strength testing, 76-mm (3-in.) diameter by

152-mm (6-in.) long cylinders with gauge studs embedded at each end for sulfate exposure testing, and 102-mm (4-in.) diameter by 204-mm (8-in.) long cylinders for compressive strength and permeability testing. The sets of specimens made for each batch are outlined in Table 3.8. The curing procedures shown in the final column and the set numbering system are explained later in this section. Single-use plastic molds were used for casting all the specimens. For the 76-mm x 152-mm (3-in. x 6-in.) cylinders with embedded gauge studs, 102-mm (4-in.) square acrylic plastic plates were used to set the studs in place. The plates were threaded such that the gauge studs could be held in place during placement and then the plate could be removed during demolding. Gauge studs were set such that a gauge length of approximately 127 mm (5 in.) existed from embedded stud end-to-end. The regular 76-mm x 152-mm (3-in. x 6-in.) cylinders and the cylinders with gauge studs were placed and consolidated using the external vibration procedures of ASTM C192-95. The 102-mm x 204-mm (4-in. x 8-in.) cylinders were placed and consolidated using the rodding procedures of ASTM C192-95.

All the specimens were initially cured in the same manner. The specimens were kept at 23 ± 3 °C (73 ± 5 °F) and covered with wet burlap and plastic bags. For the 0.35 water-to-cementitious material ratio batches, specimens were kept in this condition for 24 ± 4 hours and then demolded. These batches had quick strength development and could be demolded early. All other batches were demolded after 48 ± 4 hours to ensure enough bond had developed between the concrete and the gauge studs for demolding. After demolding, most of the specimens were transferred to a curing tank containing saturated limewater kept at 23 ± 1 °C (73 ± 3 °F). Two 76-mm x 152-mm (3-in. x 6-in.) cylinders were kept out for compressive strength evaluations. The amount of time the rest of the specimens were kept in the room temperature curing tank depended on the curing procedure being used.

Four different curing procedures were used in Phase 2 testing. Some specimens were cured following the guidelines established in the USBR 4908 Method B procedures. This involved moist curing the specimens at room temperature until a concrete compressive strength of 24.1 MPa (3,500 psi) or more was reached. Pairs of the 76-mm x 152-mm (3-in. x 6-in.) cylinders were tested following the procedures of ASTM C39-94 (Ref 45) until an

average strength of 24.1 MPa (3,500 psi) or higher was reached. When this strength was established, sulfate exposure cylinders were removed from the curing tank and immersed in a sulfate solution. The specimens cured in this manner were designated as Set #1 specimens.

Table 3.8 – Specimen Sets for Phase 2 Batches

Testing Procedure	Specimen Type	Set #	# of Cyl.	Curing Procedure*
Early Compressive Strength	3-in. x 6-in. cylinder	1	6-12	73 °F until 24.1 MPa (3,500 psi)
28-day & 6-month Compressive Strength	4-in. x 8-in. cylinder	2	2	73 °F for 28 D
		3	2	73 °F for 7 D + 100 °F for 21 D
		4	2	73 °F for 6 months
Sulfate Exposure Test	3-in. x 6-in. cylinder w/ gauge studs	1	4	73 °F until 24.1 MPa (3,500 psi)
		2	3	73 °F for 28 D
		3	3	73 °F for 7 D + 100 °F for 21 D
		4	2	73 °F for 6 months
Permeability Testing	4-in. x 8-in. cylinders	2	3	73 °F for 28 D
		3	3	73 °F for 7 D + 100°F for 21 D
		4	2	73 °F for 6 months
<p>*All specimens capped with airtight lids and cured under plastic bags and wet burlap at approximately 73 °F for the first 24 to 48 hours. Specimens are then demolded and moist cured in limewater curing tanks at the designated temperatures and for the designated times shown in the column.</p> <p>** Conversions: 1 in. = 25.4 mm; 73 °F = 23 °C; and 100 °F = 38 °C</p>				

Set #2 specimens were cured in the 23 ± 1 °C (73 ± 3 °F) room temperature curing tank up to an age of 28 days. This is a typical curing procedure that is used to establish 28-day compressive strength and permeability values for a concrete. At 28 days after mixing, sulfate exposure cylinders were immersed in a sulfate solution, 102-mm x 204-mm (4-in. x 8-in.) cylinders were tested for compressive strength, and permeability cylinders were sliced and tested for permeability.

Set #3 specimens were cured in the same 28-day period as the Set #2 cylinders. The difference was Set #3 cylinders were cured under a much higher temperature for the last 3 weeks of this period. After 7 days of moist curing at 23 ± 1 °C (73 ± 3 °F), specimens were transferred to a curing tank containing saturated limewater kept at 38 ± 3 °C (100 ± 5 °F). The tank was kept in an environmental chamber that was maintained at 38 ± 3 °C (100 ± 5 °F) and heating coils were used to ensure the water temperature in the tank remained at the high temperature level. The specimens were cured at this 38 ± 3 °C (100 ± 5 °F) temperature for 21 days. At 28 days after mixing, sulfate exposure cylinders were immersed in a sulfate solution, 102-mm x 204-mm (4-in. x 8-in.) cylinders were tested for compressive strength, and permeability cylinders were sliced and tested for permeability. The goal of this accelerated curing procedure was to develop specimens that demonstrate the permeability of concrete cured at room temperature for 6 months in the shorter time span of 28 days. Ozyildirim performed research at the Virginia Transportation Research Council demonstrating that the high temperature curing accurately simulated the long-term curing of 6 months at room temperature (Ref 46). In their low-permeability specifications for transportation facilities, the Virginia Department of Transportation specified the accelerated curing procedure for preparing concrete for rapid permeability testing (Ref 46). The purpose of the Set #3 specimens is to evaluate the accuracy of this curing method.

Set #4 specimens provide the long-term curing data with which the Set #3 specimens' results can be compared. No Set #4 specimens will have been tested by the time of this report because of the long curing period. Set #4 specimens will be kept in the room temperature curing tank for 6 months. Sulfate exposure cylinders will then be transferred to a sulfate solution and compressive and permeability specimens will be tested. The results can be compared with Set #3 results to evaluate Ozyildirim's research.

3.3.4 *Sulfate Exposure Testing Procedures*

Procedures for sulfate exposure testing follow the basic guidelines of USBR 4908 Method B using modifications suggested in test programs performed by Freeman (Ref 12) and Tikalsky (Ref 7) for The University of Texas at Austin. Method B of the USBR 4908

test procedures requires a 10 percent sodium sulfate (Na_2SO_4) solution be used for sulfate exposure. One liter of the solution was prepared by first dissolving 100 grams of anhydrous technical grade Na_2SO_4 salt in 900 milliliters of deionized water. The solution was then diluted to 1 liter using deionized water. The sodium sulfate solution was always made at least 1 day before use and kept at 23 ± 3 °C (73 ± 5 °F). Solutions were prepared such that a solution-to-specimen volume ratio of 4.08 was maintained. This ratio meets the 4 ± 0.5 ratio requirement of the USBR procedures.

When curing procedures were completed, sulfate exposure cylinders were transferred to the sodium sulfate exposure solution containers. Several 13-mm (1/2-in.) diameter PVC pipes were used to support the cylinders and prevent specimen contact with the container sides. Both the length and the mass of the cylinders were evaluated over time in the sulfate solution. Initial measurements were not taken until the specimens had soaked for 7 days. This eliminated the effects of initial absorption and the resulting initial expansion that occurs when the cylinders are first exposed to the solution (Ref 12).

The lengths of the cylinders were measured using a length comparator with a dial gauge that read to the nearest 0.001 mm (0.0001 in.). A 168.275-mm (6.6250-in.) reference bar made of a low coefficient of thermal expansion steel alloy was used for zeroing the dial gauge. The mass of the cylinders was measured using a digital scale with an accuracy of 0.1 grams. Before any measurements were taken, specimens were removed from the sulfate solution and blotted to a saturated-surface dry (SSD) condition. The SSD condition was considered to be reached when the glossy, moist appearance of the cylinder surface faded. As mentioned before, initial measurements were taken after 1 week of soaking. Measurements are required every month for 6 months after these initial measurements and then every 3 months thereafter. The final measurements should be 2 years after the initial measurements were taken. Solutions were replaced and containers cleaned during each measurement to ensure a pH below 9.75 was maintained for the sulfate environment.

This sulfate exposure testing provides data for the percent expansion over time of concrete cylinders immersed in a sodium sulfate solution. Data are also produced indicating the percent mass change that occurred in the cylinder over time. Finally, physical

observations can be made and recorded concerning crack development, spalling, and surface scaling in the specimens.

3.3.5 *Permeability Testing Procedures*

The permeability of concrete tested in Phase 2 was measured following the procedures of the rapid permeability test, ASTM C1202-94. Upon completion of curing, 102-mm x 204-mm (4-in. x 8-in.) cylinders were removed from the curing tanks and cut using an abrasive saw into 51-mm (2-in.) thick, 102-mm (4-in.) diameter concrete slices. For each room-temperature cured, accelerated cured, and long-term cured set of specimens, four slices were or will be tested.

At the end of the rapid permeability test, coulomb values representing the total current passed through the concrete slices over the 6-hour testing period were obtained. These coulomb values have been shown to be representative of the chloride ion penetrability and thus an indirect indication of the permeability of the concrete. For Phase 2 testing, permeability tests were run for concrete cured for 28 days and will be run for concrete cured for 6 months. For each specimen set, the average coulomb value for the four tested slices was recorded.

Chapter Four: Test Results

4.1 Introduction

The following chapter presents all test results of Phase 1 testing and test results to date of Phase 2 testing. For Phase 1 testing, the fresh properties, compressive strength with time, and bar expansions for each mortar are presented. For Phase 2 testing, the fresh batch properties, compressive strength development, 28-day permeability results, and initial expansion and mass change results are presented. Results for Phase 2 testing will be updated in future reports for this TxDOT project.

4.2 Phase 1 Test Results

The following sections provide the results of Phase 1 testing. First, the fresh properties of the mortar mixes are presented. The water content adjustments needed for the mortars containing pozzolans or slag are discussed, and the flow numbers obtained for each mix are given. The compressive strength developments of each mortar are then presented. Finally, the mortar bar expansions are shown.

4.2.1 *Mix Results and Fresh Mortar Properties*

A total of twenty-four mortar mixes were made for Phase 1 testing. Table 4.1 provides an outline of the batches made; it includes the batch names, the materials they contain, mix conditions, and the flow number for each mix. Most of the flow numbers were kept within the range of 100 to 120 and/or within plus or minus 10 of the equivalent plain portland cement mortar flow. The change in water content needed to achieve these flows in the mortars with a combination of cement and a mineral admixture depended on the type and amount of admixture used. For mixes containing the Class C fly ash, the water content generally needed to be reduced 3 to 6 percent from the original content used for the portland cement mortars. Mixes containing the Class F fly ash needed a 1 to 3 percent reduction. Mixes containing the slag required an increase in water as the content was increased 3 to 8 percent. As far as each cement type, the Type I cement produced the highest flow number

mortar. The Type I-II cements produced moderate flows, and the Type V cement produced the lowest flow.

Table 4.1 – Description, Mix Conditions, and Fresh Properties of Phase 1 Mortars

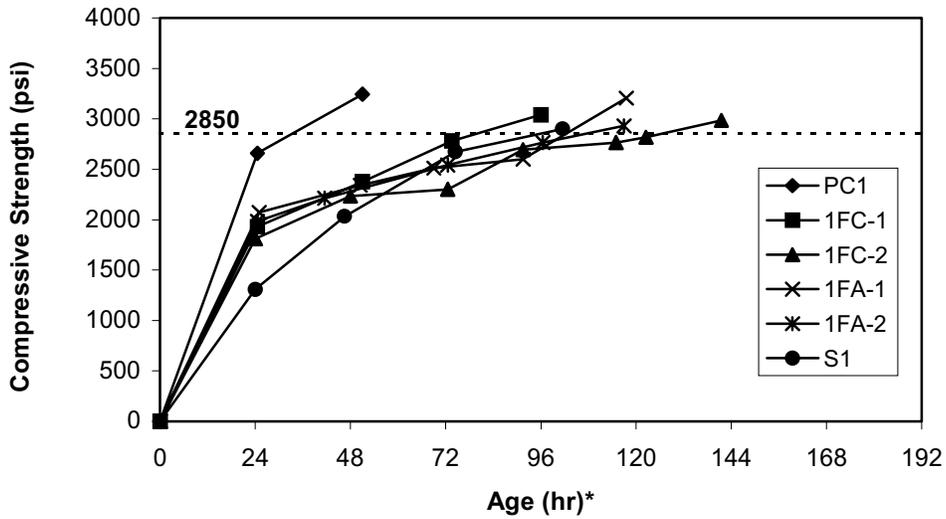
Batch Name	Cement Type	Fly Ash/Slag (% Rep.) *	Mix Temp. (°F) **	Relative Humidity (%)	Flow Number
PC1	I-II (A)	None	72	63	105
1FC-1	I-II (A)	C (25%)	73	64	109.5
1FC-2	I-II (A)	C (35%)	75	70	116
1FA-1	I-II (A)	F (20%)	73	65	107.5
1FA-2	I-II (A)	F (30%)	72	66	117.5
S1	I-II (A)	Slag (50%)	71	65	105.5
PC2	I-II (B)	None	72	62	107.5
2FC-1	I-II (B)	C (25%)	73	65	121
2FC-2	I-II (B)	C (35%)	72	67	109
2FA-1	I-II (B)	F (20%)	73	65	106
2FA-2	I-II (B)	F (30%)	72	67	113
S2	I-II (B)	Slag (50%)	75	67	114.5
PC5	V	None	75	65	97.5
5FC-1	V	C (25%)	74	65	96
5FC-2	V	C (35%)	73	63	114
5FA-1	V	F (20%)	72	62	112
5FA-2	V	F (30%)	73	62	114.5
S5	V	Slag (50%)	75	65	116
P1	I	None	73	63	120
1C-1	I	C (25%)	76	62	113.5
1C-2	I	C (35%)	71	64	119.5
1F-1	I	F (20%)	71	63	121.5
1F-2	I	F (30%)	70	62	119.5
Slag-1	I	Slag (50%)	70	60	123

* In this column, C stands for ASTM Class C fly ash, and F stands for ASTM Class F fly ash.
 ** Conversion: °C = 5/9(°F – 32)

4.2.2 Compressive Strength Results

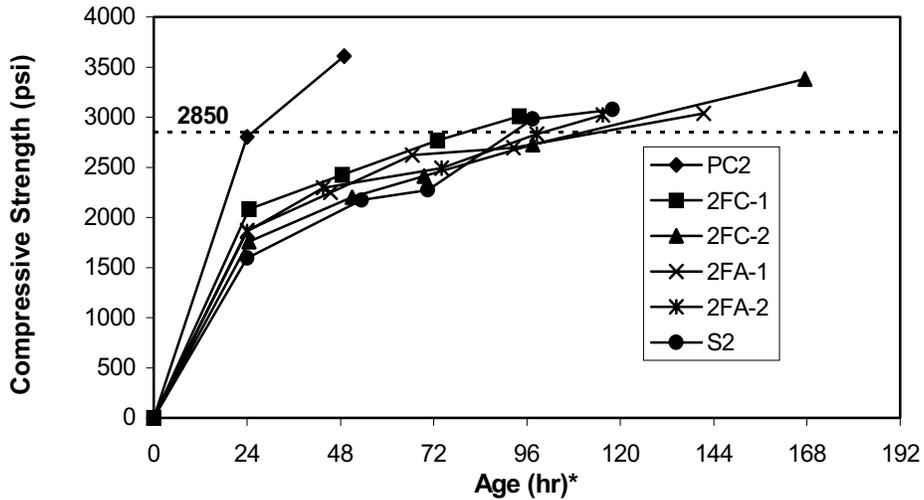
Figures 4.1, 4.2, 4.3, and 4.4 show the compressive strength development of the Phase 1 mortars for the four different cements. Table 4.2 shows the compressive strengths of

the mortars at the time of immersion in the sulfate solution and the age of the mortars at immersion. Appendix C provides complete results in tabular form.



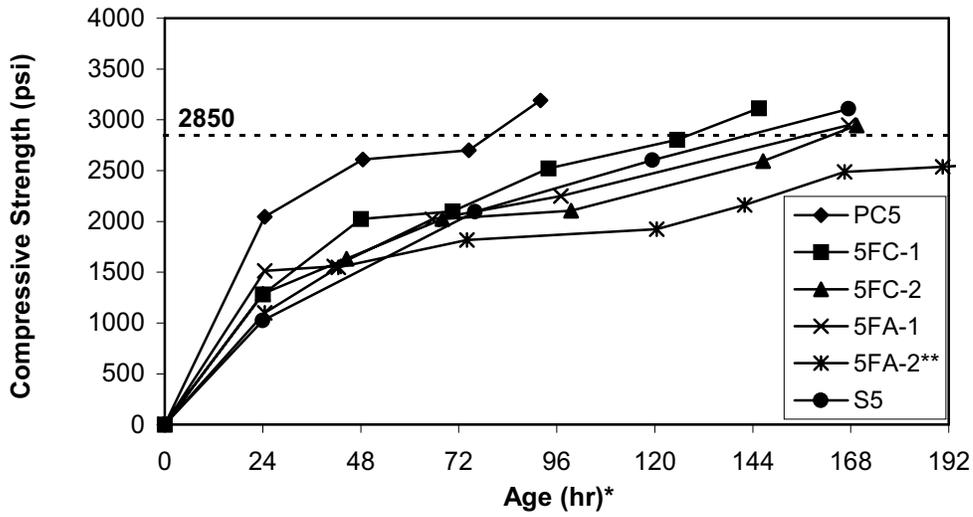
*Cured at $95 \pm 5^\circ\text{F}$ for first 24 hours and at room temperature for remainder of time
 (Conversion: 1 psi = 0.006895 MPa)

Figure 4.1 – Compressive Strengths of Mortars with Type I-II (A) Cement



* Cured at $95 \pm 5^\circ\text{F}$ for first 24 hours and at room temperature for remainder of time
 (Conversion: 1 psi = 0.006895 MPa)

Figure 4.2 – Compressive Strengths of Mortars with Type I-II (B) Cement

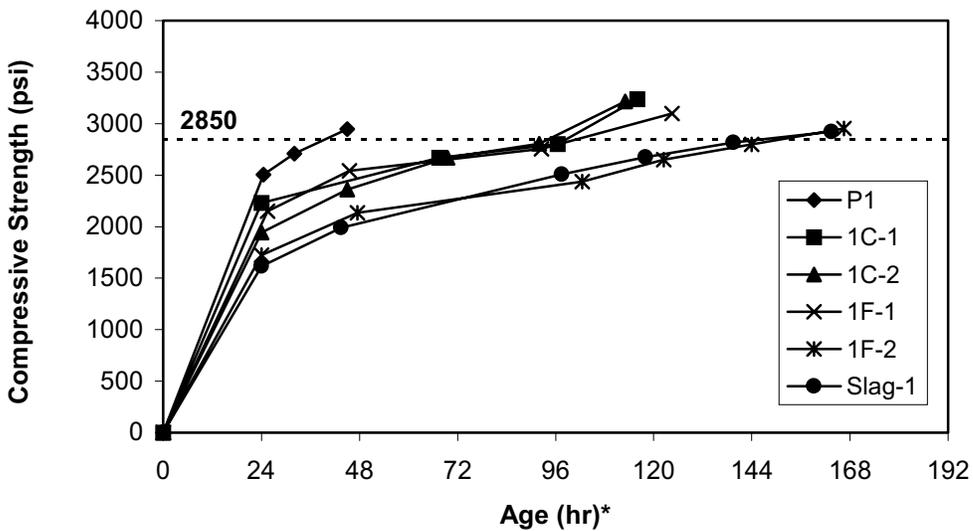


* Cured at $95 \pm 5^\circ\text{F}$ for first 24 hours and at room temperature for remainder of time

** 5FA-2 reached 2,860 psi after 11.9 total days of curing

(Conversion: 1 psi = 0.006895 MPa)

Figure 4.3 – Compressive Strengths of Mortars with Type V Cement



* Cured at $95 \pm 5^\circ\text{F}$ for first 24 hours and at room temperature for remainder of time

(Conversion: 1 psi = 0.006895 MPa)

Figure 4.4 – Compressive Strengths of Mortars with Type I Cement

Table 4.2 – Compressive Strengths and Age of Phase 1 Mortars at Immersion in Sodium Sulfate Solution

Batch Name	Cement Type	Fly Ash/Slag (% Rep.)*	At Immersion into Sulfate Solution	
			Age (days)	Compressive Strength (psi)**
PC1	I-II (A)	None	2.1	3,250
1FC-1	I-II (A)	C (25)	4	3,040
1FC-2	I-II (A)	C (35)	5.9	2,980
1FA-1	I-II (A)	F (20)	4.9	3,210
1FA-2	I-II (A)	F (30)	4.9	2,930
S1	I-II (A)	Slag (50)	4.2	2,900
PC2	I-II (B)	None	2.0	3,610
2FC-1	I-II (B)	C (25)	3.9	3,010
2FC-2	I-II (B)	C (35)	7.0	3,380
2FA-1	I-II (B)	F (20)	5.9	3,040
2FA-2	I-II (B)	F (30)	4.8	3,020
S2	I-II (B)	Slag (50)	4.9	3,080
PC5	V	None	3.8	3,190
5FC-1	V	C (25)	6.1	3,110
5FC-2	V	C (35)	7.1	2,940
5FA-1	V	F (20)	7.0	2,950
5FA-2	V	F (30)	11.9	2,860
S5	V	Slag (50)	7.0	3,100
P1	I	None	1.9	2,950
1C-1	I	C (25)	4.8	3,240
1C-2	I	C (35)	4.7	3,220
1F-1	I	F (20)	5.2	3,100
1F-2	I	F (30)	6.9	2,950
Slag-1	I	Slag (50)	6.8	2,920

* In this column, C stands for ASTM Class C fly ash and F stands for ASTM Class F fly ash.
** Conversion: 1 psi = 0.006895 MPa

Initial compressive strengths were measured when mortar specimens were removed from the 35 °C (95 °F) chamber approximately 24 hours after the specimens were cast. No mortar had reached the 19.7-MPa (2,850-psi) compressive strength plateau after this initial

24 hours of curing, and thus, all the batches required additional curing time in the room temperature saturated limewater curing tanks.

For each cement type, the plain portland cement mortars reached strengths of 19.7 MPa (2,850 psi) or higher much quicker than the mortars with mineral admixtures. The plain Type I-II cement mortars gained strength the quickest as each reached strengths above 20.7 MPa (3,000 psi) after approximately 1 day in the room temperature limewater. The mortar with the Type I-II (A) portland cement had a strength of 22.4 MPa (3,250 psi) after approximately 48 hours of total curing while the mortar with the Type I-II (B) cement had a strength of 24.9 MPa (3,610 psi) after the same amount of curing. The Type I cement had comparable early strength gain to the Type I-II cements; it reached a compressive strength of 20.3 MPa (2,950 psi) in less than 48 hours. The Type V cement mortar, PC5, had the slowest strength development. It took nearly 96 total hours of curing before a strength of 22.0 MPa (3,190 psi) was reached.

The mortars with mineral admixtures required much more curing time as no mortar reached a 19.7-MPa (2,850-psi) compressive strength before 72 hours of total curing. The mortars with combinations of the Type I-II or Type I cements with a mineral admixture usually reached 19.7-MPa (2,850-psi) compressive strengths between 4 and 7 days of curing. The Type V cement mortars with mineral admixtures usually required more curing time, and one mortar required 12 days of curing before reaching the 19.7-MPa (2,850-psi) strength plateau.

4.2.3 *Sulfate Expansion Results for Plain Portland Cement Mortars*

Detailed sulfate expansion results for all the Phase 1 mortars are provided in Appendix D. Figure 4.5 and Table 4.3 outline the expansion results for the mortars containing plain portland cement. Figure 4.5 shows measured bar expansions after 1, 2, 3, 4, 8, 13, and 15 weeks and 4 months (120 days) and 6 months (180 days) of continuous soaking in the sodium sulfate solution. For each mortar, Table 4.3 provides the average 180-day expansions along with the C_3A content of the cement tested, the number of bars tested, and the standard deviation of the data. The expansion limits defined for ASTM C1012-95 are

depicted graphically in Figure 4.5 by horizontal gridlines. According to the limits, mortars having 180-day expansions of less than 0.05 percent meet the requirements for a severe sulfate environment; mortars with a 180-day expansion of 0.10 or less meet the requirements for a moderate sulfate environment; and mortars with 180-day expansions exceeding 0.10 percent are only applicable in mild environments.

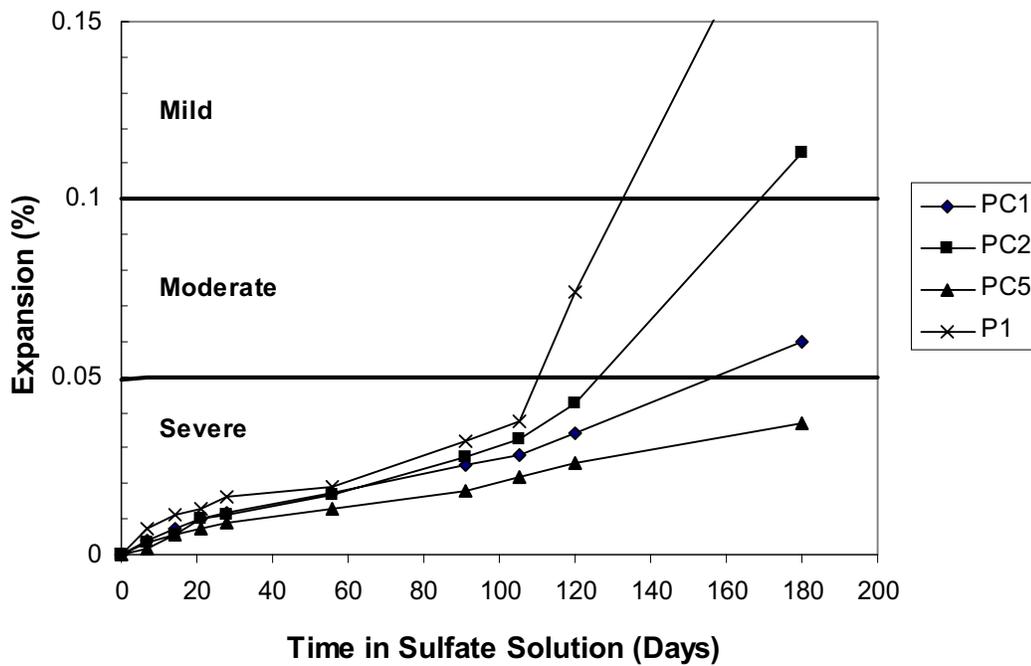


Figure 4.5 – Sulfate Expansions of Plain Portland Cement Mortars

Table 4.3 – 180-Day Sulfate Expansions of Plain Portland Cement Mortars

Batch Name	Cement Type	C ₃ A Content, %	No. of Bars	Standard Deviation, %	Average 180-Day Expansion, %
PC1	I-II (A)	5.1	6	0.004	0.060
PC2	I-II (B)	7	6	0.008	0.113
PC5	V	0	6	0.001	0.037
P1	I	12	6	0.018	0.199

The mortar bars containing the Type I-II (A) cement had average 180-day expansions slightly above 0.05 percent; an average 180-day expansion of 0.060 percent was obtained for the mortar. Data ranged from 0.054 to 0.067 percent for the six bars tested. The Type I-II (B) cement mortar had higher expansions exceeding 0.10 percent at 180 days; the average 180-day expansion for the mortar was 0.113 percent. Data ranged from 0.103 to 0.124 percent for the six bars tested. The mortar mix, PC5, for the Type V cement had the lowest average 180-day expansion; the 0.037 percent average expansion was well below 0.05 percent. The variance between the six PC5 bar specimen expansions was minimal as expansions ranged from 0.035 to 0.039 percent. The mortar mix, P1, containing the Type I cement had the highest expansions of the plain portland cement mortars as an average 180-day expansion of 0.199 percent was obtained. The 180-day expansions for all six bars were well above 0.10 percent as data ranged from 0.171 to 0.226 percent.

4.2.4 *Sulfate Expansion Results for Mortars with Mineral Admixtures*

Figures 4.6, 4.7, 4.8, and 4.9 and Tables 4.4, 4.5, 4.6, and 4.7 present the expansion results for the mortars containing combinations of cement with a mineral admixture. A figure is provided for each cement presenting the expansions for the five mortars with different cement-mineral admixture combinations. The expansion of the plain portland cement mortar is also presented in each figure such that an observation can be made as to whether the mineral admixture increased or decreased sulfate resistance. The tables provide information about the mineral admixture and the percent volumetric replacement that was used in each mix. The tables then summarize the data for the bar expansions at 180 days. For some of the mixes, 180-day expansion data were not collected for all six bars. This reduction in the number of test specimens was either because bars broke or cracked during demolding or displayed erratic expansion during the test period. For some mortars, no 180-day expansion data were collected because the bars for these batches had become so deteriorated after 180 days of soaking that they were not measurable.

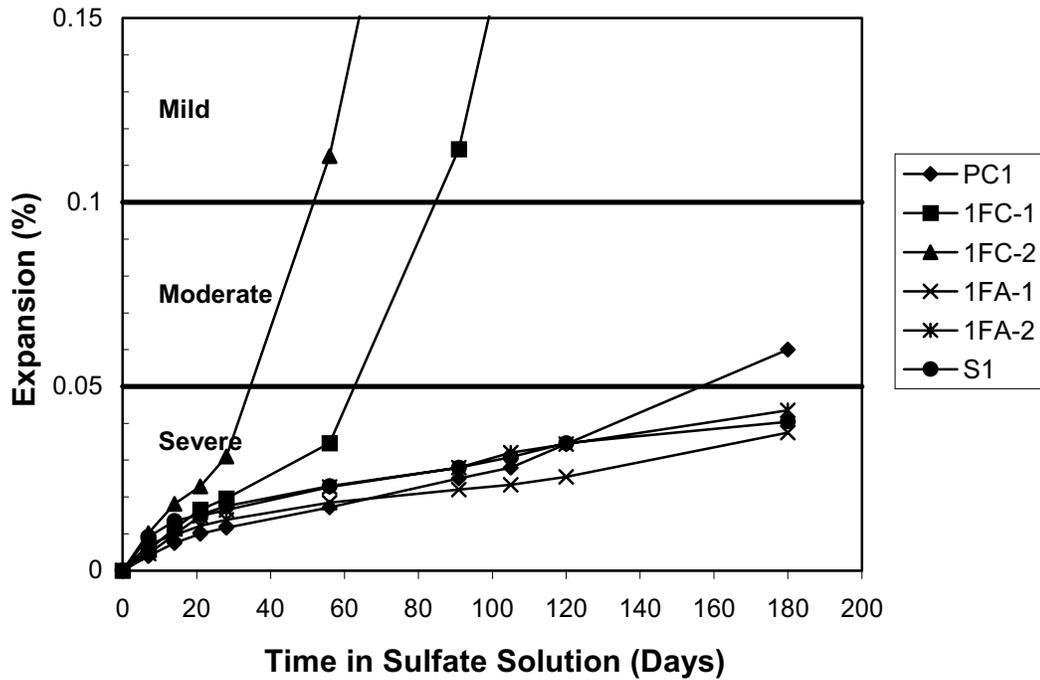


Figure 4.6 – Sulfate Expansions of Mortars Containing Combinations of Type I-II (A) Cement with Mineral Admixtures

Table 4.4 – 180-Day Sulfate Expansions of Mortars with Combinations of Type I-II (A) Cement with Mineral Admixtures

Batch Name	Fly Ash/Slag *	% Rep.	No. of Bars	Standard Deviation, %	Average 180-Day Expansion, %
PC1	None	0	6	0.004	0.060
1FC-1	C	25	4	0.121	0.711
1FC-2	C	35	--	--	--
1FA-1	F	20	6	0.003	0.038
1FA-2	F	30	6	0.003	0.044
S1	Slag	50	5	0.002	0.040

* In this column, C denotes ASTM Class C fly ash and F denotes ASTM Class F fly ash.

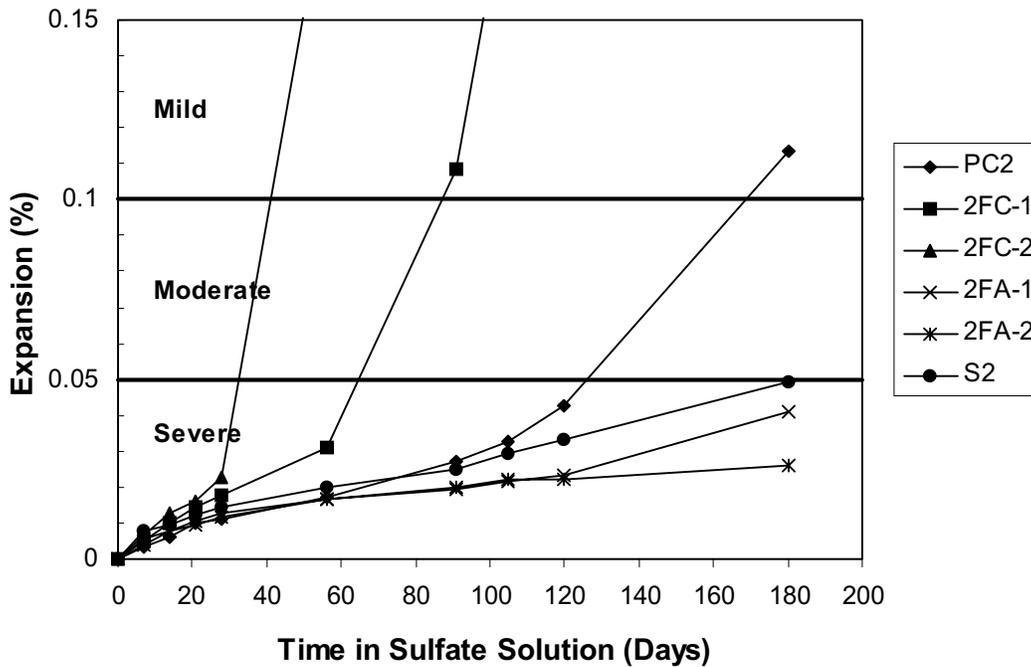


Figure 4.7 – Sulfate Expansions of Mortars Containing Combinations of Type I-II (B) Cement with Mineral Admixtures

Table 4.5 – 180-Day Sulfate Expansions of Mortars with Combinations of Type I-II (B) Cement with Mineral Admixtures

Batch Name	Fly Ash/Slag *	% Rep.	No. of Bars	Standard Deviation, %	Average 180-Day Expansion, %
PC2	None	0	6	0.008	0.113
2FC-1	C	25	6	0.219	0.974
2FC-2	C	35	--	--	--
2FA-1	F	20	6	0.004	0.041
2FA-2	F	30	6	0.001	0.026
S2	Slag	50	6	0.007	0.049

* In this column, C denotes ASTM Class C fly ash and F denotes ASTM Class F fly ash.

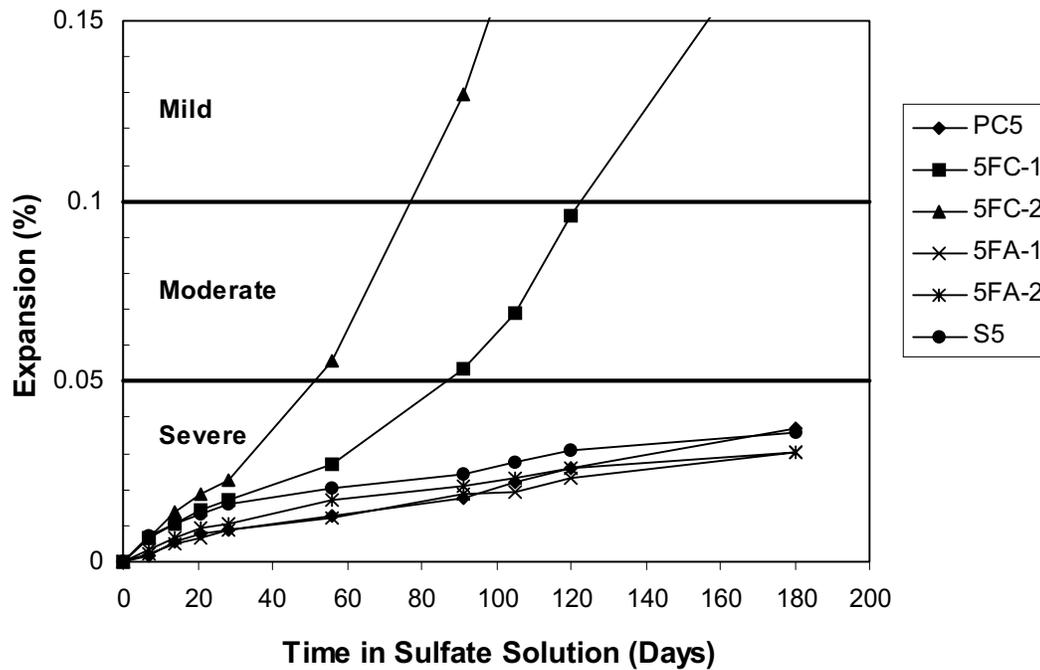


Figure 4.8 – Sulfate Expansions of Mortars Containing Combinations of Type V Cement with Mineral Admixtures

Table 4.6 – 180-Day Sulfate Expansions of Mortars with Combinations of Type V Cement with Mineral Admixtures

Batch Name	Fly Ash/Slag *	% Rep.	No. of Bars	Standard Deviation, %	Average 180-Day Expansion, %
PC5	None	0	6	0.001	0.037
5FC-1	C	25	5	0.039	0.185
5FC-2	C	35	6	0.034	0.459
5FA-1	F	20	5	0.001	0.030
5FA-2	F	30	6	0.002	0.030
S5	Slag	50	5	0.001	0.036

* In this column, C denotes ASTM Class C fly ash and F denotes ASTM Class F fly ash.

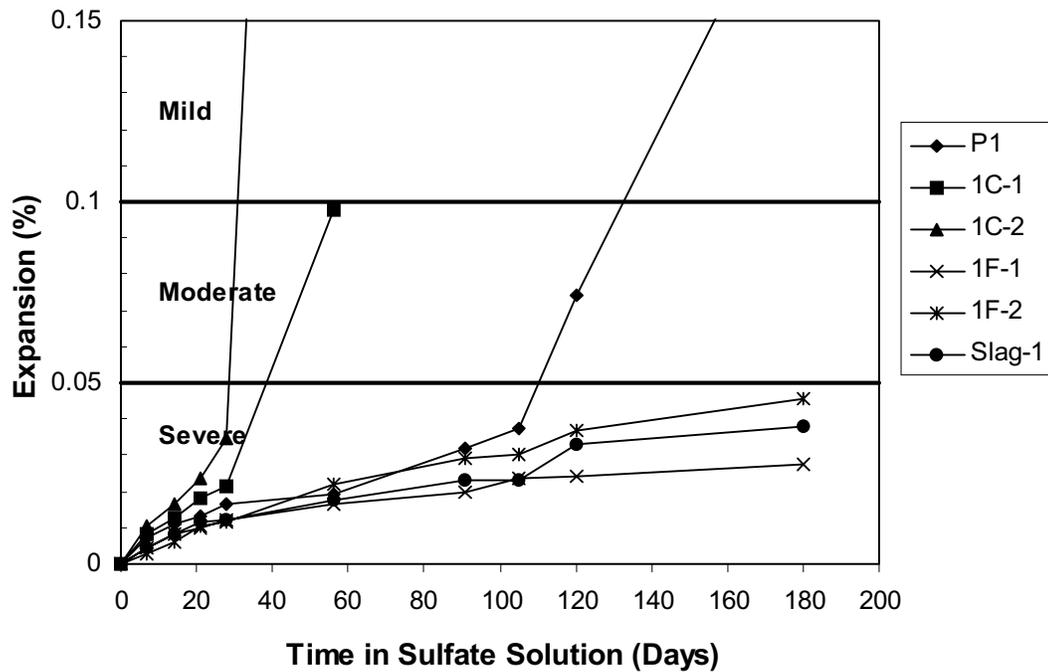


Figure 4.9 – Sulfate Expansions of Mortars Containing Combinations of Type I Cement with Mineral Admixtures

Table 4.7 – 180-Day Sulfate Expansions of Mortars with Combinations of Type I Cement with Mineral Admixtures

Batch Name	Fly Ash/Slag *	% Rep.	No. of Bars	Standard Deviation, %	Average 180-Day Expansion, %
P1	None	0	6	0.018	0.199
1C-1	C	25	--	--	--
1C-2	C	35	--	--	--
1F-1	F	20	6	0.001	0.028
1F-2	F	30	5	0.001	0.045
Slag-1	Slag	50	6	0.002	0.038

* In this column, C denotes ASTM Class C fly ash and F denotes ASTM Class F fly ash.

The effect mineral admixtures had on the sulfate resistance of mortars varied depending on the cement with which they were blended. In general, all mortar mixes containing the ASTM Class C fly ash reached expansions exceeding 0.10 percent or failed because of cracking before 180 days of soaking. Mortars containing the ASTM Class F fly ash and the GGBF slag all had average 180-day bar expansions below 0.05 percent. Another consistent pattern was that the standard deviation between bar expansions within a mortar specimen set typically decreased as the 180-day expansions of the mortar decreased.

For the Type I-II (A) cement, the mortars containing the Class F fly ash and the slag significantly reduced expansions as the 0.060 percent average 180-day expansion of plain portland cement mortar was reduced 28 to 38 percent with the addition of the admixtures. A 20 percent volumetric replacement of the cement with the Class F fly ash resulted in the highest reduction while a 30 percent replacement with the fly ash provided the lowest reduction. Data for all batches were very consistent as the range between bar expansions within a specimen set never exceeded 0.008 percent, and the standard deviation was 0.003 percent or lower.

The 180-day expansion reductions by the Class F fly ash and the slag were even greater for mortars containing the Type I-II (B) cement as the 0.113 percent average 180-day expansion of the plain portland cement mortar was reduced 57 to 77 percent. The 2FA-1 mix containing a 20 percent volumetric replacement of the Class F fly ash reduced the plain cement mortars' 180-day expansion 64 percent to an average of 0.041 percent. Expansion reductions were even more dramatic when a replacement of 30 percent was used for this fly ash; the 2FA-2 mix reduced expansions 77 percent to an average 180-day expansion of 0.026 percent. The S2 mix containing slag reduced average expansions 57 percent; the 180-day average expansion for this mix was 0.049 percent. The consistency of the data for these mixes was good as standard deviations ranged from 0.001 to 0.007 percent.

The reductions in 180-day expansions provided by the Class F fly ash and slag were relatively small when the admixtures were used in combination with the Type V cement. With 20 percent Class F fly ash replacement, mix 5FA-1 had an average 180-day expansion of 0.030 percent reducing the expansion of the plain portland cement mortar by 19 percent.

Mix 5FA-2 with a 30 percent fly ash replacement showed the same improvement as the same average 180-day expansion was obtained. The mix containing the slag, S5, provided little improvement as an average expansion of 0.036 percent was found at 180 days. This value is only 0.001 percent less than the plain cement mortar expansion. All of these mixes had consistent data as standard deviations were 0.002 percent or lower.

The most drastic level of reductions in sulfate expansions provided by the Class F fly ash and slag came when the admixtures were used with the Type I cement. The 1F-1 with a 20 percent Class F fly ash replacement provided the greatest improvement as a 180-day expansion of 0.028 percent was obtained. This was a reduction of 86 percent from the expansion found for the plain cement mortar. The 1F-2 mix with a 30 percent Class F fly ash reduced expansions 77 percent as a 180-day average expansion of 0.045 percent was obtained. The slag mix, Slag-1, reduced expansions 81 percent as a 180-day average expansion of 0.038 percent was measured.

Mortars containing the Type I-II (A) cement combined with the Class C fly ash had extremely high expansions as the mortar, 1FC-1, that contained a 25 percent volumetric replacement of the Class C fly ash had an average 180-day expansion of 0.711 percent. Expansions were erratic as some bars were not measurable after 180 days of soaking and the standard deviation between expansions of the measurable bars was 0.121 percent. A 35 percent replacement with the fly ash resulted in even higher expansions such that the bars for the mortar, 1FC-2, expanded at a faster rate than the 1FC-1 bars before becoming severely deteriorated and unmeasurable after 180 days of soaking. Figure 4.10 shows the condition of the 1FC-2 mortar bars after 13 weeks of soaking.

The use of the Class C fly ash with the Type I-II (B) cement resulted in the same dramatic increases in expansions observed earlier. The mix with the Type I-II (B) cement and 25 percent fly ash replacement, 2FC-1, had bar expansions as high as 1.3 percent as an average 180-day expansion of 0.974 percent was obtained. This high value is nearly 9 times the expansion of the plain cement mortar. The mix with 35 percent fly ash replacement expanded at an extremely high rate as all bars had become unmeasurable after 13 weeks of soaking. Figure 4.11 shows the mortar bars for the 2FC-2 mix after 13 weeks of soaking.

The bars were in even worse condition than those shown in Figure 4.10 for the 1FC-2 mix. Some bars were completely deteriorated, and most were severely warped and cracked.



Figure 4.10 – 1FC-2 Mortar Bars after 13 Weeks of Sulfate Solution Soaking

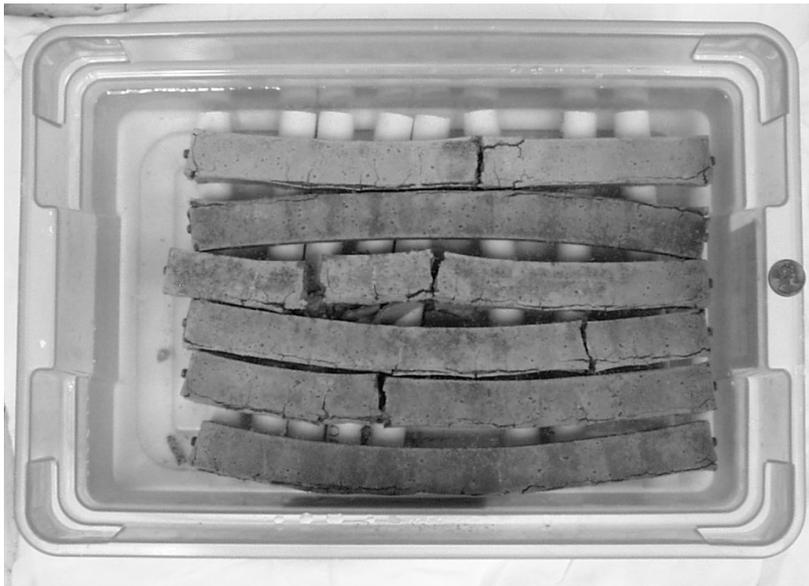


Figure 4.11 – 2FC-2 Mortar Bars after 13 Weeks of Sulfate Solution Soaking

The use of the Class C fly ash with the Type V cement once again resulted in dramatic expansion increases as the 5FC-1 mix containing a 25 percent fly ash replacement expanded 5 times as much as the plain cement mortar after 180 days of soaking. The 5FC-2 mix with a 35 percent replacement expanded approximately 12 times as much as an average expansion of 0.459 was obtained. Neither mix displayed the deterioration nor cracking that was typical for other mixes containing the Class C fly ash.

The combination of Class C fly ash with a Type I cement resulted in mortars that deteriorated quickly as all bars for the 1C-1 mix containing 25 percent of the fly ash became unmeasurable after 8 weeks of soaking. The bars for the 1C-2 mix with 35 percent of the fly ash became unmeasurable between 4 and 8 weeks of soaking. Figure 4.12 shows the condition of the 1C-2 bars after 8 weeks of soaking in sulfate solution.



Figure 4.12 – 1C-2 Mortar Bars after 8 Weeks of Sulfate Solution Soaking

4.3 Phase 2 Test Results

The following sections provide information on the mix results, compressive strength development, permeability results, initial expansions, and initial mass changes for the concrete batches made for Phase 2 testing. Results for the long-term, 6-month cured

specimens will not be provided in this report. These results along with the long-term expansions and mass changes of the all the test mixes will be provided in future reports.

4.3.1 Mix Results and Fresh Concrete Properties

A total of twenty-two concrete batches were made for Phase 2 testing. Table 4.8 outlines the batch names, mix conditions, and fresh concrete properties.

Table 4.8 – Description, Mix Conditions, and Fresh Properties of Phase 2 Concretes

Batch Name	Cement Type	Fly Ash* (% Rep.)	W/C Ratio (by weight)	HRWR Admixture (fl oz/100 lb cement)	Mix Temp. (°F)	R.H. (%)	Conc. Temp. (°F)	Slump (in.)
PC1-1	I	None	0.35	15	68	45	72	2
PC1-2	I	None	0.45	7	72	53	75	3.5
PC1-3	I	None	0.55	--	74	52	75	8
PC2-1	I-II (B)	None	0.35	10	70	44	72	1
PC2-2	I-II (B)	None	0.45	2	55	65	64	5
PC2-3	I-II (B)	None	0.50	--	67	31	70	7
PC2-4	I-II (B)	None	0.55	--	68	31	67	8
PC5-1	V	None	0.35	18	72	58	75	1
PC5-2	V	None	0.45	5	60	30	64	4
PC5-3	V	None	0.55	--	70	58	72	8
2FC-1a	I-II (B)	C (25)	0.35	12	80	65	80	1
2FC-2a	I-II (B)	C (25)	0.45	1	70	66	74	3
2FC-3a	I-II (B)	C (25)	0.55	--	70	66	73	8.5
2FC-1b	I-II (B)	C (35)	0.35	12	70	44	70	2.5
2FC-2b	I-II (B)	C (35)	0.45	1	59	45	65	3.5
2FC-3b	I-II (B)	C (35)	0.55	--	61	45	63	8
2FA-1a	I-II (B)	F (20)	0.35	15	70	28	67	4
2FA-2a	I-II (B)	F (20)	0.45	2	70	38	70	2.5
2FA-3a	I-II (B)	F (20)	0.55	--	70	38	70	8
2FA-1b	I-II (B)	F (30)	0.35	18	73	25	70	1
2FA-2b	I-II (B)	F (30)	0.45	2	69	30	72	4.5
2FA-3b	I-II (B)	F (30)	0.55	--	70	45	72	8

* In this column, C stands for ASTM Class C fly ash and F stands for ASTM Class F fly ash.
 Conversions: °C = 5/9(°F – 32); 1 in. = 25.4 mm

4.3.2 *Compressive Strength Results*

Table 4.9 outlines the compressive strength development of the batches made for Phase 2 testing. Detailed results are provided in Appendix E. Early compressive strength results were found by testing pairs of 76-mm x 152-mm (3-in. x 6-in.) cylinders. The table provides the age at which the concrete strength was found to be at or above 24.1 MPa (3,500 psi) and the strength of the concrete when it was immersed in the sulfate solution. The table also provides the 28-day compressive strengths of the batches found by testing 102-mm x 204-mm (4-in. x 8-in.) cylinders. The average 28-day compressive strengths of room-temperature cured and accelerated cured specimens are given.

For the batches with plain portland cement, the concretes with the Type I-II (B) cement typically displayed the fastest strength development and the highest ultimate 28-day strengths. The Type I cement concrete strengths were usually slightly lower while the Type V cement concretes always had the slowest strength development and usually displayed the lowest ultimate strengths. The only exception was the 0.35 water-to-cement ratio concrete containing the Type V cement. This batch had the highest 28-day room-temperature cured compressive strength of all the plain portland cement concretes. Compressive strength development always slowed and ultimate strengths always decreased as the water-to-cement ratios of the mixes increased. The accelerated cured specimens that were moist cured for 7 days at 23°C (73°F) and then moist cured 21 days at 38°C (100°F) typically had 28-day compressive strengths slightly higher than the room-temperature cured specimens. Strength increase ranged from 0 to 9 percent. The greatest strength difference was in the 0.35 water-to-cement ratio mix containing the Type I-II (B) cement as the accelerated cured specimens had an average strength 4.8 MPa (700 psi) higher than the room-temperature cured specimens. There were two unusual cases with the 0.45 and 0.55 water-to-cement ratio concretes containing the Type V cement, where the accelerated cured specimens actually had strengths lower than the room-temperature cured specimens.

Table 4.9 – Compressive Strength Development for Phase 2 Concretes

Batch Name	Cement Type	Fly Ash* (% Rep.)	W/C Ratio (by weight)	At Immersion in Sulfate Solution		28-Day Compressive Strength (psi)**	
				Age (days)	Strength (psi)**	Room Temp. Cured	Accelerated Cured
PC1-1	I	None	0.35	1.3	4110	6960	7440
PC1-2	I	None	0.45	2	4230	6320	6620
PC1-3	I	None	0.55	8	3620	5150	5490
PC2-1	I-II (B)	None	0.35	1.1	4820	7980	8680
PC2-2	I-II (B)	None	0.45	2	3740	6670	7120
PC2-3	I-II (B)	None	0.50	3.1	3610	5990	6060
PC2-4	I-II (B)	None	0.55	6.0	3570	5250	5450
PC5-1	V	None	0.35	3.8	5420	8170	8440
PC5-2	V	None	0.45	4.8	3610	5620	5470
PC5-3	V	None	0.55	28	4020	4020	4000
2FC-1a	I-II (B)	C (25)	0.35	1.0	3950	8620	9420
2FC-2a	I-II (B)	C (25)	0.45	3.0	4020	6410	6900
2FC-3a	I-II (B)	C (25)	0.55	8.0	3690	4960	5230
2FC-1b	I-II (B)	C (35)	0.35	2.2	4680	8610	9600
2FC-2b	I-II (B)	C (35)	0.45	3.8	4060	6990	7660
2FC-3b	I-II (B)	C (35)	0.55	9.0	3660	5180	5900
2FA-1a	I-II (B)	F (20)	0.35	2.0	4850	8480	9630
2FA-2a	I-II (B)	F (20)	0.45	3.3	4100	6380	7980
2FA-3a	I-II (B)	F (20)	0.55	14.9	3620	5080	5870
2FA-1b	I-II (B)	F (30)	0.35	2.0	3580	7010	8220
2FA-2b	I-II (B)	F (30)	0.45	4.8	3830	5830	6760
2FA-3b	I-II (B)	F (30)	0.55	11.9	3700	4560	5870

* In this column, C stands for ASTM Class C fly ash and F stands for ASTM Class F fly ash.
 ** Conversion: 1 psi = 0.006895 MPa

Compressive strength developments for the concretes containing combinations of the Type I-II (B) portland cement with the ASTM Class F and Class C fly ashes were generally slower than for the plain Type I-II (B) portland cement concretes. The concrete mixes containing the Class C fly ash generally developed strength faster than the mixes containing the Class F fly ash as the 0.55 water-to-cementitious material ratio mixes containing the

Class F fly ash took a particularly long time to reach the 24.1-MPa (3,500-psi) strength plateau.

The cement–mineral admixture combination concretes with water-to-cementitious material ratios of 0.45 and 0.55 always had moist, room-temperature cured, 28-day compressive strengths that were lower than the plain portland cement concretes with the same water-to-cementitious material ratios. The mixes with a 0.35 water-to-cementitious material ratio generally had moist-cured 28-day strengths higher than the plain portland cement concretes. The only exception was the 2FA-lb batch that contained a 30 percent volumetric replacement of the Class F fly ash. This concrete had a low strength of 48.3 MPa (7,010 psi) that was 6.7 MPa (970 psi) lower than the plain portland cement concrete with a 0.35 water-to-cement ratio. In fact, all the concretes containing a 30 percent replacement of Class F fly ash had fairly low 28-day moist-cured strengths.

The level of the 28-day compressive strengths for the accelerated cured cement–mineral admixture combination concretes depended on the type of fly ash. Accelerated cured 28-day strengths for the concretes with the ASTM Class C fly ash were lower than the plain portland cement concrete strengths when water-to-cementitious material ratios of 0.45 and 0.55 were used and were higher when a water-to-cementitious material ratio of 0.35 was used. When the Class F fly ash was used, all accelerated cured, 28-day compressive strengths were higher than the accelerated cured strengths of the equivalent plain portland cement concretes when a 20 percent volumetric replacement level was used while strengths were lower when a 30 percent replacement level was used. All concretes with Class F fly ash had significant strength increases when comparing accelerated curing and room-temperature curing as strength increases as high as 11.0 MPa (1,600 psi) occurred. Accelerated cured specimens were as much as 25 percent stronger than room-temperature cured specimens in some cases.

4.3.3 *28-Day Permeability Results*

Table 4.10 displays 28-day permeability results for all the concrete batches made for Phase 2 testing. Average coulombs passed for the room-temperature cured specimens and the accelerated cured specimens are given.

Table 4.10 – 28-Day Permeability Results for Phase 2 Concretes

Batch Name	Cement Type	Fly Ash* (% Rep.)	W/C Ratio (by weight)	Room Temperature Cured (28 D @ 73 °F)**	Accelerated Cured (7D @ 73 °F + 21D @ 100 °F)**
				Avg. Coulombs Passed	Avg. Coulombs Passed
PC1-1	I	None	0.35	2,170	1,610
PC1-2	I	None	0.45	3,490	2,810
PC1-3	I	None	0.55	6,020	4,990
PC2-1	I-II (B)	None	0.35	2,240	1,710
PC2-2	I-II (B)	None	0.45	4,110	3,210
PC2-3	I-II (B)	None	0.50	4,410	3,820
PC2-4	I-II (B)	None	0.55	5,610	4,800
PC5-1	V	None	0.35	2,310	1,830
PC5-2	V	None	0.45	4,030	3,350
PC5-3	V	None	0.55	5,000	4,310
2FC-1a	I-II (B)	C (25)	0.35	1,360	730
2FC-2a	I-II (B)	C (25)	0.45	3,030	1,790
2FC-3a	I-II (B)	C (25)	0.55	5,500	3,380
2FC-1b	I-II (B)	C (35)	0.35	1,790	800
2FC-2b	I-II (B)	C (35)	0.45	2,390	1,130
2FC-3b	I-II (B)	C (35)	0.55	5,350	2,510
2FA-1a	I-II (B)	F (20)	0.35	1,530	590
2FA-2a	I-II (B)	F (20)	0.45	2,540	960
2FA-3a	I-II (B)	F (20)	0.55	4,010	1,590
2FA-1b	I-II (B)	F (30)	0.35	1,160	430
2FA-2b	I-II (B)	F (30)	0.45	1,540	570
2FA-3b	I-II (B)	F (30)	0.55	2,460	880

* In this column, C stands for ASTM Class C fly ash and F stands for ASTM Class F fly ash.
 ** Conversion: °C = 5/9(°F - 32)

For the plain portland cement concretes, average 28-day room-temperature cured specimens' coulombs passed values ranged from 2,170 to 6,020 as permeability increased with increasing water-to-cement ratios. The permeability values were fairly independent of the type of cement that was used. Average 28-day accelerated cured specimen coulombs passed values ranged from 1,610 to 4,990 as the accelerated curing led to reductions in permeability ranging from 13 to 26 percent. The percent reductions in permeability owing to

accelerated curing were generally higher when the water-to-cement ratio of the concrete was lower.

For the Phase 2 concretes containing Class C fly ash, average 28-day room-temperature cured specimen coulombs passed values were always lower than the coulomb values for the plain portland cement concretes with the same water-to-cementitious material ratios. The reductions were typically highest for the 0.35 water-to-cementitious material ratio concretes as the concrete containing a 25 percent volumetric replacement of the Class C fly ash had a coulomb value of 1,360 that was 39 percent lower than the plain portland cement concrete's value. Accelerated curing reduced these coulomb values significantly as coulomb values dropped between 39 and 55 percent when compared to the normal, room-temperature cured values. The reduction in permeability owing to accelerated curing was again generally higher when the water-to-cement ratio of the concrete was lower.

Most Phase 2 concretes containing the Class F fly ash had coulomb values that were lower than the plain portland cement concretes and the Class C fly ash–cement combination concretes with the same type of curing and water-to-cementitious material ratio. The 28-day average coulombs passed values for room-temperature cured specimens ranged from 1,160 to 4,010 while accelerated cured specimens' coulomb values ranged from 430 to 1,590 with the lowest permeabilities being found for the concretes containing a 30 percent volumetric replacement of the Class F fly ash. Accelerated curing significantly reduced coulombs passed values as reductions ranged between 60 to 63 percent.

4.3.4 *Initial Sulfate Expansion and Mass Change Results*

The first three sets of sulfate exposure specimens from each Phase 2 batch are currently immersed in sulfate solution and being monitored for expansion and mass change. Appendix F provides the initial expansion and mass change results for all Phase 2 concretes including the expansion and mass changes of the 24.1-MPa (3,500-psi) USBR cured cylinders; the 28-day room-temperature cured cylinders; and the 28-day accelerated cured cylinders. This Appendix provides results obtained through April 1999. Results are presented in both tabular and graphical form. It is difficult to interpret much from these early

results because no specimen has been exposed long enough to the sulfate solution to display any significant expansion or mass change. Past research where the USBR 4908 Method B procedures were used showed that it requires between 180 to 270 days of exposure before any delineation can be made between the performance of one set of specimens versus another (Ref 7). Measurements of expansions and mass changes will continue through an exposure period of approximately 18 months.

Chapter Five: Discussion and Analysis

5.1 Introduction

The test program for this report consisted of a two-phase investigation. In Phase 1, a performance test for sulfate resistance was evaluated. In Phase 2, the effects of concrete mix properties and curing conditions on permeability and the effects of permeability on concrete sulfate resistance were investigated. The overall purpose of the test program was the establishment of performance-based specifications for the durability of concrete against sulfate exposure. The preceding chapter presented the results obtained thus far from this test program. In this chapter, these results will be analyzed and their implications concerning specification development will be discussed.

5.2 Discussion of Phase 1 Test Results

In Phase 1, the mortar bar expansion test, ASTM C1012-95, “Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution,” was used to evaluate the effects of portland cement chemistry and mineral admixtures on sulfate resistance (Ref 1). The purpose of the testing was to evaluate the suitability of ASTM C1012-95 as a performance test. The following sections discuss the experimentally determined effects of the different material properties and the correlation of these results with past experience within the concrete industry. The consistency of the test data and the effectiveness of the expansion criteria established by ASTM Subcommittee C01.29 for assessing levels of sulfate resistance are also evaluated. Analysis is used to assess the adequacy of ASTM C1012-95 as a performance test for sulfate resistance and to develop recommendations concerning incorporating the test into performance-based specifications.

5.2.1 *Effects of Cement Chemistry on Sulfate Resistance*

As discussed in Chapter 2, past research has established that the most important portland cement chemical component affecting sulfate resistance is the tricalcium aluminate (C_3A) content of the cement. In Phase 1 testing, four cements with C_3A contents ranging

from 0 to 12 percent were tested. Figure 5.1 provides a plot of the average 180-day expansions that were obtained for the four plain portland cement mortars versus the C₃A contents of the cements used in each mortar.

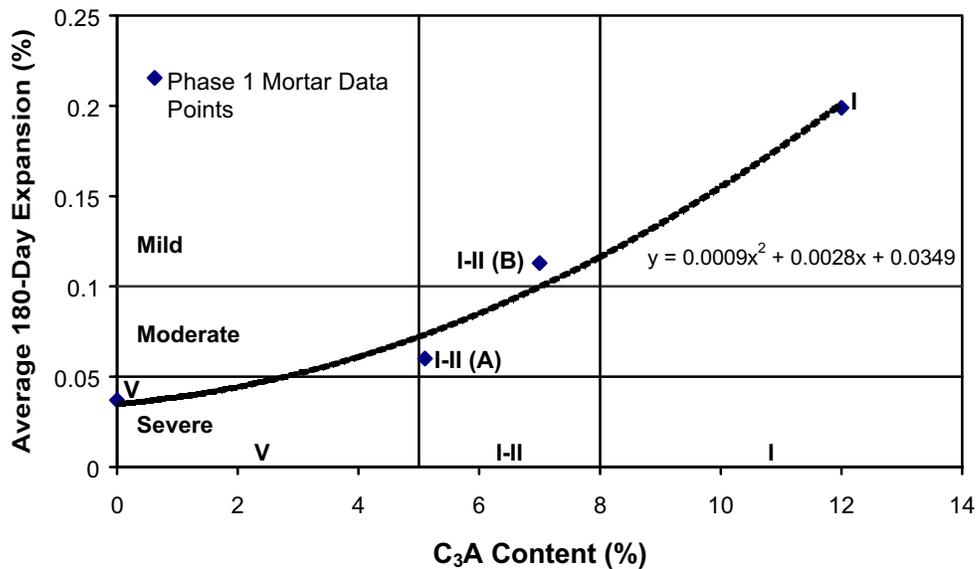


Figure 5.1 – Average 180-Day Expansions versus Cement C₃A Contents for Plain Portland Cement Mortars

The horizontal gridlines in Figure 5.1 represent the ASTM C1012-95 expansion limits established by ASTM Subcommittee C01.29 and proposed in this project’s performance-based specifications. The line at a 180-day expansion of 0.1 percent delineates between mild and moderate sulfate environment resistance while the line at an expansion of 0.05 percent delineates between moderate and severe sulfate resistance. The vertical gridlines represent the portland cement C₃A content limits established in the ASTM C150 specifications for categorizing sulfate resistant cements. A line is provided at 5 percent C₃A to represent the maximum allowable C₃A content of severe sulfate-resistant Type V cements, and another line is provided at 8 percent to represent the maximum allowable content for the moderate sulfate resistant Type I-II or Type II cements.

Phase 1 testing of plain portland cement mortars confirmed the well-supported fact that the C_3A content of a cement greatly impacts its sulfate resistance. The second-degree polynomial trendline shown in Figure 5.1 that was developed using the four Phase 1 test data points displays a clear increase in expansion and thus decrease in sulfate resistance as the cement C_3A content increased. The rate of expansion increased as the C_3A contents increased.

In comparing the levels of sulfate resistance determined for the four Phase 1 portland cements using the ASTM C1012-95 expansion criteria versus the ASTM C150 C_3A content limits, three out of the four cement evaluations came up with the same results. The Type I cement was found to be only adequate in mild sulfate environments according to ASTM C1012-95 because of its high average 180-day expansion of 0.199 percent and according to ASTM C150 because of its high C_3A content of 12 percent. The Type V cement proved to be adequate for severe sulfate environments according to ASTM C1012-95 because its average 180-day expansion was a low 0.037 percent and according to ASTM C150 because of its C_3A content of 0 percent. The Type I-II (A) cement met the requirements for moderate sulfate environments according to ASTM C1012 because of its average 180-day expansion of 0.060 percent and according to ASTM C150 because of its C_3A content of 5.1 percent.

The one case where ASTM C1012 sulfate resistance evaluations did not match ASTM C150 evaluations was for the Type I-II (B) cement. The cement had a C_3A content of 7 percent that placed it within the category of a moderate sulfate resistant cement according to ASTM C150 specifications. However, tests on the plain portland cement mortar containing the Type I-II (B) cement produced an average 180-day expansion of 0.113 percent. This expansion is above 0.1 percent and thus makes the cement inadequate for moderate sulfate environments according to ASTM C1012-95 expansion criteria.

The high expansion of the Type I-II (B) cement mortar is not altogether surprising when past research is considered. First of all, when C_3A content limits for cement were first being researched in 1949 by the Portland Cement Association, the value of 7 percent C_3A was established as the borderline between good and poor cement performance against sulfate attack. According to this research, the Type I-II (B) cement evaluated in Phase 1 testing may

be right on the border between being a mild or moderate sulfate resistant cement. Secondly, among the several different cement types tested in developing the expansion limits for ASTM C1012, no Type I-II or Type II cement with a C_3A content higher than 6.8 percent was evaluated (Refs 30, 31). The only tested cements with C_3A contents between 7 and 8 percent were blended cements where the C_3A content was estimated using the Bogue method (Ref 31). There is no evidence showing plain portland cement mortars with cement C_3A contents between 7 and 8 percent had 180-day expansions of less than 0.10 percent. Finally, other components of a cement's chemistry beyond the C_3A content affect its sulfate resistance. For borderline cements like the Type I-II (B) cement tested in this program, these components, such as gypsum, could push the cement into a lower category of sulfate resistance.

According to Phase 1 test results, especially those concerning the Type I-II (B) cement, the expansion criteria established for ASTM C1012-95 can be conservative in comparison to the ASTM C150 specifications when used to determine portland cement sulfate resistance levels. The second-degree equation corresponding to the trendline shown in Figure 5.1 can be used to calculate 180-day expansion criteria that directly correspond with ASTM C150 limits according to Phase 1 test results. According to the trendline equation, an average 180-day expansion of 0.071 percent corresponds to a C_3A content of 5 percent and is, thus, the borderline between severe and moderate sulfate resistance. An average 180-day expansion of 0.115 percent corresponds to a C_3A content of 8 percent and is, thus, the borderline between moderate and mild sulfate resistance. The expansion values are 0.021 and 0.015 percent higher than the ASTM Subcommittee C01.29 expansion limits. Results indicate that borderline cements with C_3A contents between 7 and 8 percent that are considered moderate sulfate resistant cements according to ASTM C150 specifications may be categorized as mild sulfate resistant cements according to current ASTM C1012-95 expansion limits. Also, ASTM C150 Type V severe sulfate resistant cements with C_3A contents between 4 and 5 percent may be found to only meet moderate sulfate resistance requirements when ASTM C1012-95 is used.

Owing to the limited amount of data collected in Phase 1, it would be unreasonable to recommend an increase in ASTM C1012 180-day sulfate expansion limit values based on these results. A more realistic interpretation of the results is that they provide a warning to engineers that caution should be taken when using borderline cements with C_3A contents between 4 and 5 percent and 7 and 8 percent. The cement may have a high gypsum content or other characteristic that may push the cement to a lower category of sulfate resistant behavior. The best approach an engineer can take is to avoid the use of the borderline 4 to 5 percent C_3A content cements in severe sulfate environments and borderline 7 to 8 percent C_3A content cements in moderate sulfate environments. If this cannot be avoided, the engineer should test the borderline cement using the ASTM C1012-95 procedures and current expansion criteria to ensure the cement is adequate.

5.2.2 *Effects of Mineral Admixtures on Sulfate Resistance*

In Phase 1, the effects of an ASTM Class F fly ash, an ASTM Class C fly ash, and a ground granulated blast furnace slag (GGBFS) on the sulfate resistance of mortars were evaluated using the procedures of ASTM C1012-95. The following section discusses the effects of each mineral admixture that were revealed from the test results. Results are compared with past experience as the ultimate goal is to evaluate the accuracy of ASTM C1012-95 as a performance test for determining the sulfate resistance of cement-mineral admixture combinations.

ASTM Class F Fly Ash

Past experience has revealed that fly ashes meeting the requirements for ASTM Class F fly ash generally improve sulfate resistance. Despite the relatively high calcium content of the ASTM Class F fly ash used in Phase 1 testing, most sources show this fly ash can also be expected to improve sulfate resistance. Using the Dunstan's R factor approach to predicting fly ash performance against sulfate attack, no significant improvement in sulfate resistance is expected from the Phase 1 fly ash. The ASTM Class F fly ash CaO content of 17.3 percent and Fe_2O_3 content of 5.9 percent yield a Dunstan R factor of 2.1. This R factor puts the

Phase 1 Class F fly ash in the range of fly ashes that are expected to provide no significant change to sulfate resistance. However, the Dunstan R factor is conservative because it does not consider the impacts of silica, alumina, and sulfur content in the fly ash.

The Tikalsky-Carrasquillo procedures for predicting the fly ash effect on sulfate resistance provide a more comprehensive evaluation of the fly ash because most of the fly ash chemical components that influence sulfate resistance are considered. The CaO content of 17.3 percent for the Phase 1 Class F fly ash puts it within the range of 10 to 25 percent CaO content fly ashes that must be examined using a CaO-SiO₂-Al₂O₃ ternary diagram. Figure 5.2 shows how the glassy portions of the Class F fly ash lie in the ternary diagram. The star point is determined based on the fly ash's 47.8 percent SiO₂ content, 22.6 percent Al₂O₃ content, and 17.3 CaO content and, thus, a 54.5 percent SiO₂ glassy phase, 25.8 percent Al₂O₃ glassy phase, and 19.7 percent CaO glassy phase. The glassy portion of the ash is in the upper half of the anorthite field such that the ash is likely to improve sulfate resistance according to the Tikalsky-Carrasquillo recommended fly ash evaluation procedures.

Information supporting the Phase 1 Class F fly ashes' ability to improve sulfate resistance is also provided by Tikalsky and Carrasquillo in their evaluations of the effects of twenty different fly ashes on the sulfate resistance of concrete (Ref 7). Tikalsky and Carrasquillo revealed trends showing fly ashes with a total SiO₂, Al₂O₃, and Fe₂O₃ content exceeding 70 percent generally improved sulfate resistance. Their research also revealed fly ashes with a silica (SiO₂) content greater than 40 percent and low sulfur trioxide contents typically improved sulfate resistance. The Phase 1 Class F fly ash has a total SiO₂, Al₂O₃, and Fe₂O₃ content of 76.3 percent, a SiO₂ content of 47.8 percent, and a low sulfur trioxide content of 0.7 percent. All these chemical properties indicate this class of fly ash will improve sulfate resistance according to Tikalsky and Carrasquillo's research.

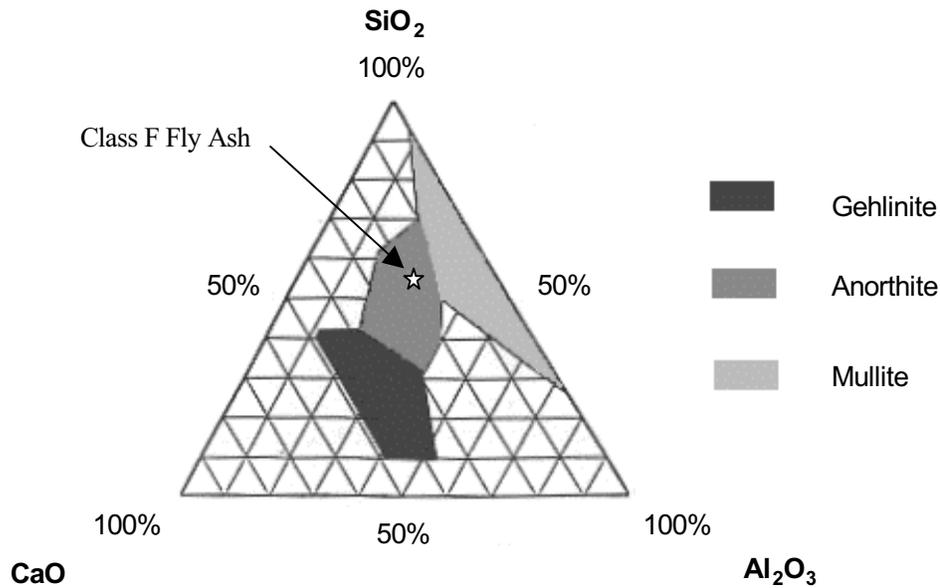


Figure 5.2 – Evaluation of Reactivity of Phase 1 Class F Fly Ash Glassy Phases Using Ternary Diagram

Results of Phase 1 ASTM C1012 testing on mortars containing the Class F fly ash confirmed that the fly ash did indeed improve sulfate resistance. Figure 5.3 shows the average 180-day sulfate expansions of all the Phase 1 mortars containing the Class F fly ash and the corresponding plain portland cement mortars. All mortars containing the Class F fly ash had improved sulfate resistance compared to their respective plain portland cement mortar. All Class F fly ash mortars had average 180-day expansions below 0.05 percent such that each mortar met the requirements for severe sulfate resistance according to ASTM C1012 expansion criteria.

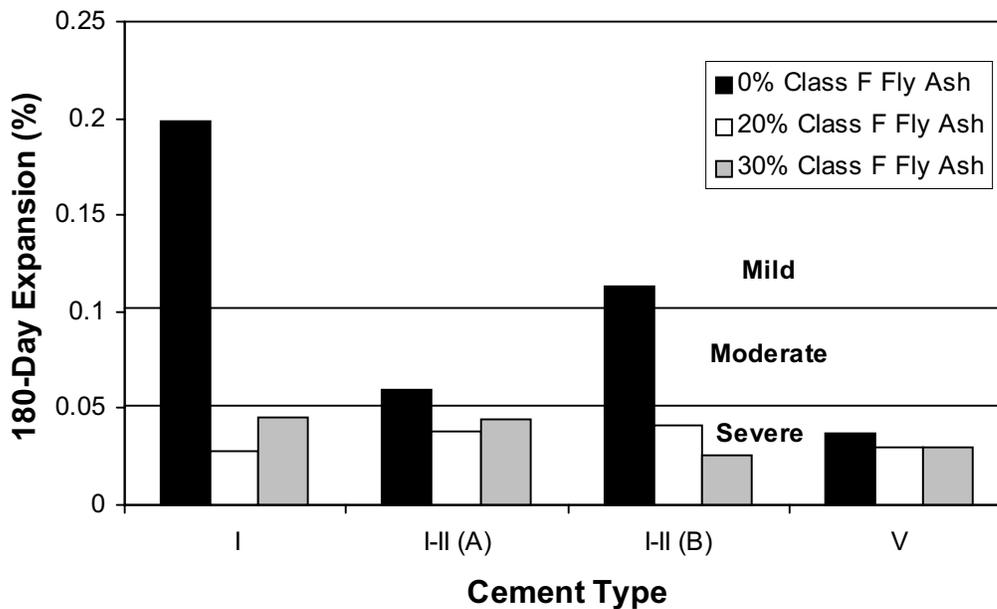


Figure 5.3 – 180-Day Sulfate Expansions of Mortars with Class F Fly Ash

The level of reductions in sulfate expansion depended on the type of cement with which the fly ash was being used; reductions increased as the C_3A content of the cement increased. Reductions in expansion were as small as 0.07 percent when the Class F fly ash was used with the Type V cement and as high as 0.171 percent when the fly ash was used with the Type I cement. No trends were observed concerning the impact of fly ash content on sulfate resistance because sometimes the 20 percent replacement mortar would have lower expansions while other times the 30 percent replacement mortar had the lower value. This lack of correlation between sulfate resistance and fly ash content for Class F fly ashes was also discovered by Tikalsky and Carrasquillo in their research (Ref 7).

ASTM Class C Fly Ash

Owing to the high calcium content of ASTM Class C fly ashes, most of these fly ashes have been shown to provide little improvement if not reductions in sulfate resistance. For the Class C fly ash evaluated for Phase 1 testing, the Dunstan and Tikalsky-Carrasquillo procedures for predicting the effect of fly ash on sulfate resistance show this fly ash should

have a significantly negative impact on the resistance of mortars to sulfate attack. With a CaO content of 27.6 percent and a Fe₂O₃ content of 5.9 percent, the Phase 1 Class C fly ash has a Dunstan's R factor of 3.8. This R factor puts the fly ash in the category of fly ashes that can be expected to reduce sulfate resistance. Tikalsky and Carrasquillo recommended that any fly ash with a CaO content exceeding 25 percent should not be used in concrete exposed to sulfate. Based on these recommendations, the Phase 1 Class C fly ash with its CaO content of 27.6 percent is not applicable in sulfate environments as the fly ash is expected to severely reduce sulfate resistance.

Phase 1 test results supported the predictions of Dunstan, Tikalsky, and Carrasquillo in that all mortars containing the Class C fly ash had expansions well above 0.1 percent before 180 days of soaking. The use of the Class C fly ash with any of the cement types yielded mortars that were unsuitable for application in environments where sulfates are a concern. All mortars either cracked and deteriorated to the point where they were unmeasurable before 180 days of soaking was reached or had extremely high expansions at 180 days.

Figure 5.4 shows the 56-day expansions of the mortars with Class C fly ash and the corresponding plain portland cement mortars. The 56-day measurement period was chosen because this was the longest period at which at least one bar from all the batches was measurable. After 56 days, sets of mortar bars with Class C fly ash started to become totally deteriorated. The results in Figure 5.4 show that Class C fly ash quickly reduced sulfate resistance and the reduction increased with increasing fly ash content. This relationship between fly ash content and sulfate resistance where increased content yields increased expansions was also observed in Tikalsky and Carrasquillo's work (Ref 7). Figure 5.4 also shows that the rate of expansion of the mortars with Class C fly ash increased as the C₃A content of the portland cement in the mortar increased. In the end, all the mortars containing Class C fly ash showed either significant deterioration or high expansions that indicated the mortars were not suitable for application in a sulfate environment.

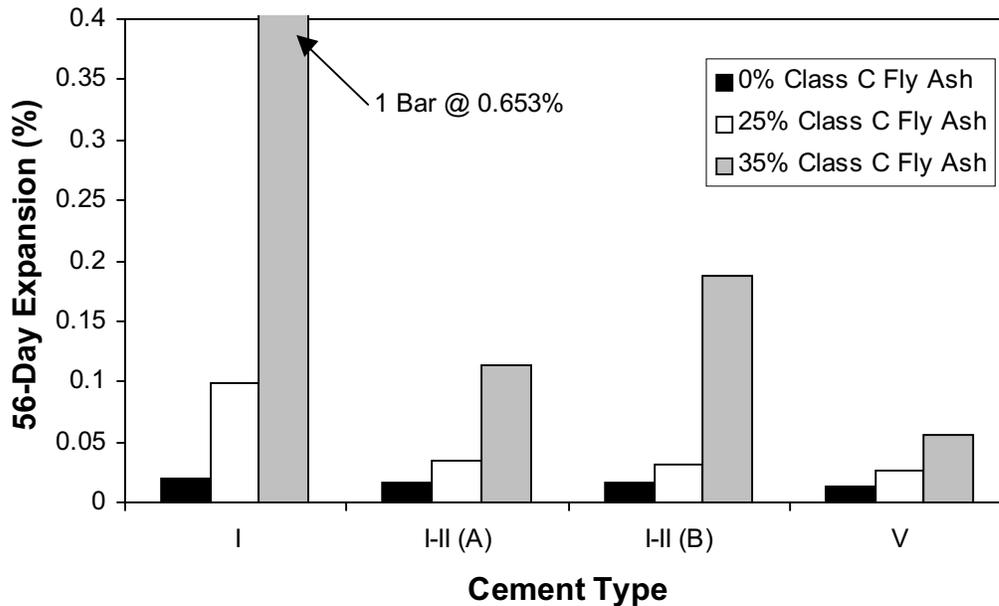


Figure 5.4 – 56-Day Sulfate Expansions of Mortars with Class C Fly Ash

Ground Granulated Blast Furnace Slag

The final mineral admixture that was evaluated in Phase 1 testing was the ground granulated blast furnace slag. Past research concerning the effects of slag on sulfate resistance indicated slag helps to improve sulfate resistance because it reduces the amount of available calcium hydroxide. Little research has been done to provide methods for predicting the effectiveness of one slag versus another except that some researchers have found that using slag at high replacement levels increases its effectiveness in improving sulfate resistance. For Phase 1 testing, a slag volumetric replacement level of 50 percent was used; this is a typical replacement value used for slag in laboratories and in the field.

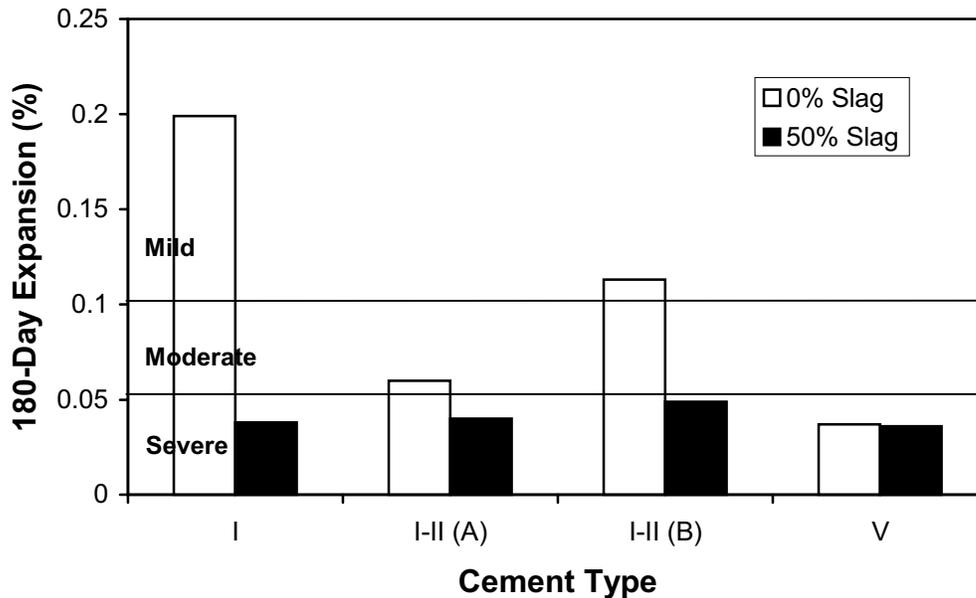


Figure 5.5 – 180-Day Sulfate Expansions of Mortars with GGBF Slag

Phase 1 test results for mortars containing slag correlated well with what would be expected from past experience. All the mortars had reduced sulfate expansions in comparison to their respective plain portland cement mortar. Figure 5.5 shows the average 180-day expansions of the mortars with slag and the plain portland cement mortars. Similar to the mortars with Class F fly ash, all the mortars containing slag had 180-day expansions less than 0.05 percent; the level of reductions in expansions increased as the C_3A content of the portland cement increased. According to the ASTM C1012-95 expansion criteria, all the mortars with slag met the requirements for severe sulfate resistance. The mortars performed as well as the mortars containing Class F fly ash. These results show the 50 percent slag volumetric replacement level was enough to provide sufficient calcium hydroxide consumption and thus improved sulfate resistance.

5.2.3 *Final Evaluation of ASTM C1012-95*

The ultimate goal of Phase 1 testing was to determine the suitability of ASTM C1012-95 as a performance test for evaluating the sulfate resistance of portland cement and portland cement-mineral admixture combinations. The test can be evaluated by investigating the following three issues:

1. The consistency, and thus the reliability, of results obtained from the test.
2. Correlation of test results with past experience in the concrete industry.
3. Accuracy and effectiveness of the test's expansion criteria used for determining the level of sulfate resistance of a mortar.

In order to be an adequate performance test, ASTM C1012-95 must be consistent, correlate with past experience, and have effective corresponding performance criteria.

The consistency and reliability of results produced from Phase 1 testing can be assessed by examining the standard deviations that were calculated for each set of mortar bar data. Figure 5.6 shows a plot of the standard deviation values calculated for several of the Phase 1 mortar data sets versus the average 180-day sulfate expansion values that were obtained for the mortars. The only mortars not included in this figure are the mortars containing Class C fly ash because these mortars either deteriorated before 180-day measurements were obtained or had unusually high expansions.

Figure 5.6 shows that the standard deviation between mortar bar expansions within a set generally increased as the 180-day expansions of the mortars increased. The Class C fly ash mortars supported this pattern. They had the highest expansions and also the highest standard deviations of all the mortars with data standard deviations ranging from 0.034 to 0.219 percent. This pattern is a positive trend because it indicates that mortars with 180-day expansions in the lower range will have lower data standard deviations and thus more consistent data. Engineers using ASTM C1012-95 will generally only be concerned with the reliability of data for mortars with 180-day expansions at or below 0.1 percent because these

are the mortars that will be labeled acceptable according to test criteria for application in a moderate or severe sulfate environment. As shown in Figure 5.6, Phase 1 mortars with expansions less than 0.1 percent generally had data standard deviations of less than 0.005 percent and many had deviations at or below 0.002 percent. Data did not get erratic until 180-day expansions started to exceed 0.1 percent. Phase 1 results indicate engineers can be confident that data obtained from ASTM C1012-95 testing will be reliable for any mortar that is categorized as a moderate or severe sulfate resistant mortar.

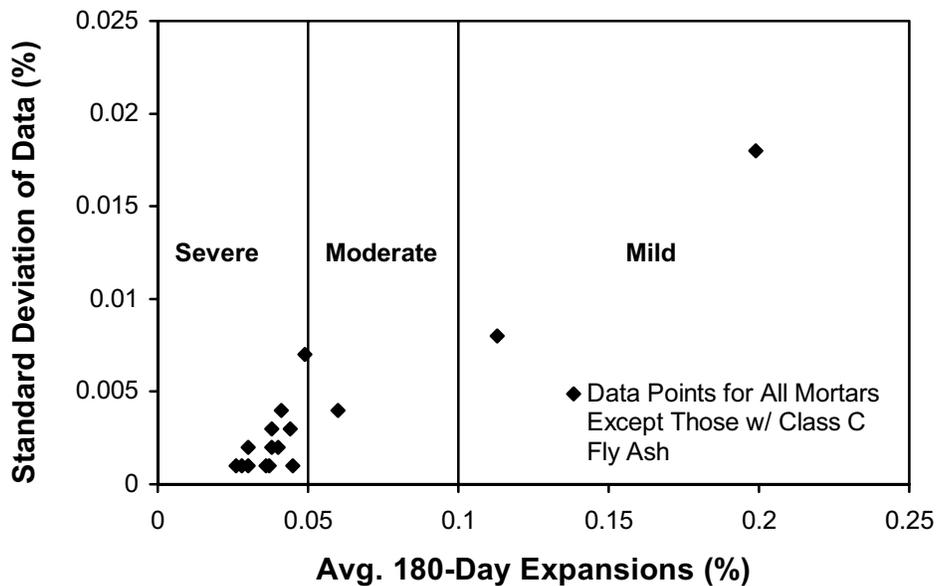


Figure 5.6 – Mortar Data’s Standard Deviation versus Average 180-Day Expansions

The ASTM C1012 test’s accuracy in predicting mortar sulfate resistances can only be determined by testing materials where knowledge of their sulfate resistance behaviors already exists and showing test results correlate with past experience. Through years of investigation, researchers and engineers have established patterns in concrete material behavior in sulfate environments. These patterns include decreasing sulfate resistance with increasing C_3A contents in portland cements, improved sulfate resistance with the use of Class F fly ash and slag, and reduced sulfate resistances with the use of Class C fly ash. One

major goal of Phase 1 testing was to test the ASTM C1012-95 test's ability to reveal these patterns. ASTM C1012-95 passed this examination and all results correlated well with what was expected based on past experience. Phase 1 testing showed ASTM C1012-95 has the ability to reveal both negative and positive characteristics in the sulfate resistances of both plain portland cements and cement-mineral admixture combinations.

The final step in evaluating ASTM C1012-95 involves examining the expansion criteria established by ASTM Subcommittee C01.29. The criteria can be evaluated by comparing them with specifications that are currently being used to delineate between mild, moderate, and severe sulfate resistant materials and ensuring the current specifications and the ASTM C1012 criteria produce similar results. The main specification currently in use for the purpose of sulfate resistance categorization is the ASTM C150 portland cement C_3A content limit that categorizes cements into Type V, severe sulfate resistant cements; Type II or I-II, moderate sulfate resistant cements; and non-sulfate-resistant Type I cements.

In section 5.2.2, a comparison between ASTM C150 specifications and ASTM C1012-95 criteria was made based on expansion results for the four Phase 1 plain portland cement mortars. The comparison indicated the specifications and the expansion criteria correlated fairly well as three out the four cements were determined to have the same level of sulfate resistance using the two indicators. For the one cement where results did not match, ASTM C1012 results indicated the cement was only adequate for mild sulfate environments while ASTM C150 specifications categorized the cement as a moderate sulfate resistant cement. This result indicates the ASTM C1012-95 expansion criteria can sometimes be conservative in comparison to ASTM C150 specifications. This possibility is not altogether negative as a conservative criteria means a safer criteria as far as assuring concrete durability against sulfate attack. Based on Phase 1 test results and past laboratory and analytical tests examining ASTM C1012 and its expansion criteria, the expansion limits of 0.05 percent 180-day expansions or less for severe sulfate resistance and 0.10 percent 180-day expansions or less for moderate sulfate resistance can be considered acceptable. The limits are accurate and safe in assessing the levels of sulfate resistance of cements or cement-mineral admixture combinations.

5.2.4 *Implications of Results Concerning Specifications*

Phase 1 results have indicated ASTM C1012-95, “Standard Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution,” is an adequate performance test for determining the level of resistance of any cementitious material against sulfate attack. The test has been shown to produce consistent and reliable results. Results correlate well with past laboratory experience. Finally, the performance criteria have shown to be effective at determining a level of sulfate resistance for a material. With these results, ASTM C1012-95 has proven to be an acceptable performance test for application in the TxDOT performance-based specifications being prepared in this project.

The proposed performance-based specifications for sulfate resistance developed earlier in this project were presented in Chapter 2. Through examining ASTM C1012 procedures, performing Phase 1 testing, and analyzing test results, a couple of issues arose where it seemed changes in the proposed specifications would make them more effective. The first issue involves eliminating the mass change limits that were incorporated into the performance criteria corresponding to ASTM C1012. As discussed in Chapter 2, it was determined early in the test program preparation that incorporating a mass change measurement into ASTM C1012 procedures would be difficult and may interfere with expansion results. Also, the mass change criterion serves as an indicator of surface scaling, but this type of damage will most likely not occur in the mortar bars that are tested in ASTM C1012-95.

A second specification-related issue arose from Phase 1 plain portland cement mortar test results, where it was found that borderline portland cements with C_3A contents between 7 and 8 percent and 4 and 5 percent had the potential of dropping a sulfate resistance level according to ASTM C1012 criteria. These results should raise caution to engineers that cements with C_3A contents between 7 and 8 percent may not be adequate for moderate sulfate environments, and cements with C_3A contents between 4 and 5 percent may not be adequate for severe sulfate environments. A safe approach for addressing this issue in the specifications would be to require engineers to test borderline cements. Any Type II or I-II cement with a C_3A content between 7 and 8 percent or any Type V cement with a C_3A

content between 4 and 5 percent should be examined using ASTM C1012-95 to assure the cements meet performance criteria.

Tables 5.1 and 5.2 show the modified performance-based specifications for sulfate resistance containing the changes discussed above. It should be noted these tables may be modified again after Phase 2 results are finalized because changes may be made with the water-to-cementitious material ratio limits and recommendations may be made concerning the use of the USBR 4908 Method B procedures for additional performance evaluations.

Table 5.1 – Mix Design Parameters for Concrete Resistant to Sulfate Attack

Severity of Environment	Cement Type	Maximum w/c
Mild	--	--
Moderate ¹	Type II ²	0.50
Severe	Type V ³	0.45
Very Severe	Type V + pozzolan or slag ⁴	0.45

- 1 When chloride or other depassivating agents are present in addition to sulfate, a lower water-cement ratio may be necessary to reduce corrosion of embedded items.
- 2 Or use a combination of cement and mineral admixture that meets or exceeds the performance of Type II cement, as tested by ASTM C 1012-95, and the performance requirements of Table 5.2. Also, Type II cements with C₃A contents between 7 and 8 percent should be examined using ASTM C1012-95 and shown to meet the performance requirements of Table 5.2.
- 3 Or use a combination of cement and mineral admixture that meets or exceeds the performance of Type V cement, as tested by ASTM C 1012-95, and the performance requirements of Table 5.2. Also, Type V cements with C₃A contents between 4 and 5 percent should be examined using ASTM C1012-95 and shown to meet the performance requirements of Table 5.2.
- 4 Use a pozzolan or slag that has been determined via ASTM C1012-95 to improve sulfate resistance when used in concrete containing Type V cement.

Table 5.2 – Performance Criteria for Sulfate Resistance by ASTM C1012-95

Performance Criteria	Resistance Level	
	Moderate	Severe
Expansion, %	0.10	0.05

5.3 Discussion of Phase 2 Test Results

For Phase 2 testing, only intermediate results are available because long-term measurements and testing remain. The following sections discuss the results that have been obtained and that were presented in Chapter 4. The effects of cement type, mineral admixtures, water-to-cementitious material ratios, and curing conditions on compressive strength and permeability are discussed. Results are compared with past experience, and initial implications concerning the performance-based specifications are considered. No discussion is provided concerning initial expansions and mass changes of the sulfate exposure specimens because it is too early for any conclusions to be drawn.

5.3.1 *Effects of Mix Properties and Curing Conditions on Compressive Strength*

The 28-day compressive strengths for Phase 2 concretes ranged between 27.6 and 66.4 MPa (4,000 and 9,630 psi) depending on the cements, mineral admixtures, and water-to-cementitious material ratios that were used in the mix. Strengths also depended on the conditions under which specimens were cured. The following paragraphs review the strengths of the plain portland cement concretes, the concretes with Class C fly ash, and the concretes with Class F fly ash as trends are examined concerning the effects of mix properties and curing conditions on compressive strength.

Figure 5.7 shows the 28-day moist, room-temperature cured compressive strengths of the ten plain portland cement concretes that were tested. Figure 5.8 shows how accelerated curing affected compressive strength as both the room-temperature cured and accelerated cured 28-day strengths (bars with dotted lines) are presented. For the plain portland cement concretes, compressive strengths all increased as water-to-cementitious material ratios decreased. The type of cement used in the concretes seemed not to have a large impact on ultimate strengths. The Type V cement concretes typically had the lowest strengths with the exception of the 0.35 water-to-cement ratio concrete mix.

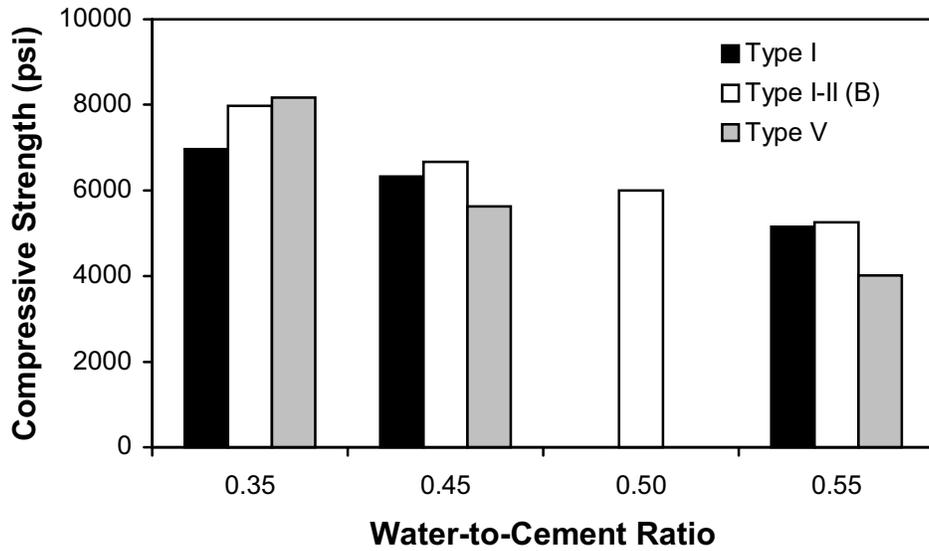


Figure 5.7 – 28-Day Room-Temperature Cured Compressive Strengths for Plain Portland Cement Concretes
(Conversion: 1 psi = 0.006895 MPa)

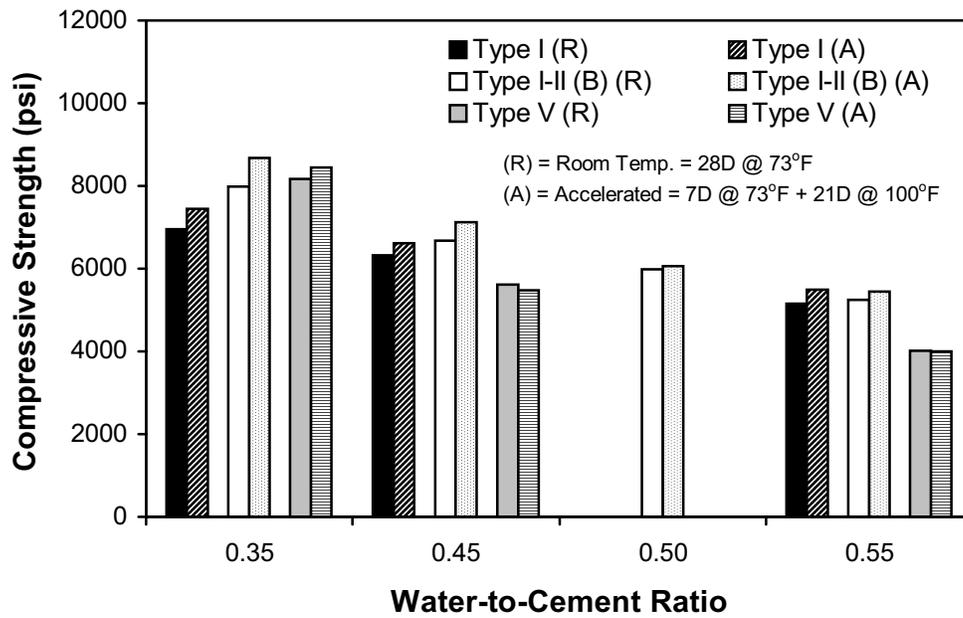


Figure 5.8 – Comparison of 28-Day Room-Temperature and Accelerated Cured Compressive Strengths for Plain Portland Cement Concretes
(Conversion: 1 psi = 0.006895 MPa)

The accelerated curing of the plain portland cement concretes generally increased strengths slightly between 0 and 9 percent when compared to the room temperature cured specimen strengths. With two of the Type V cement concretes, accelerated cured 28-day compressive strengths were actually slightly lower than room-temperature cured 28-day strengths. For the most part, it can be concluded that accelerated curing did little to improve the 28-day compressive strengths of the plain portland cement concretes. This result is not too surprising because past research has shown that plain portland cement concretes develop most of their strength capacity within the first 28 days of curing. Strength development for plain portland cement concretes after 28 days generally flattens out. Thus, the increased heat and the potential increase in the rate of strength development provided by accelerated curing will not necessarily lead to much higher strengths in plain portland cement concretes.

Figure 5.9 shows the 28-day moist, room-temperature cured compressive strengths of the six concretes containing Class C fly ash and the corresponding plain Type I-II (B) portland cement control concretes. Figure 5.10 provides a comparison of these strengths with the strengths obtained when accelerated curing procedures were used. For room-temperature curing, the 0.45 and 0.55 water-to-cementitious material ratio concretes with Class C fly ash generally had compressive strengths that were either slightly higher or slightly lower than the plain Type I-II (B) cement concrete strengths. When the low 0.35 water-to-cementitious material ratio was used, the Class C fly ash concretes had significantly higher compressive strengths than the plain concrete. While differences were typically minimal, the higher fly ash content of 35 percent typically resulted in larger strengths than the 25 percent content. It can be concluded that using Class C fly ash certainly did not reduce 28-day concrete strengths significantly because in some cases the fly ash actually significantly improved compressive strengths. This result is somewhat expected with high calcium fly ashes like the Class C fly ash used in Phase 2 testing. These fly ashes have been shown to yield 28-day compressive strengths equal to or above plain portland cement concrete strengths (Ref 7).

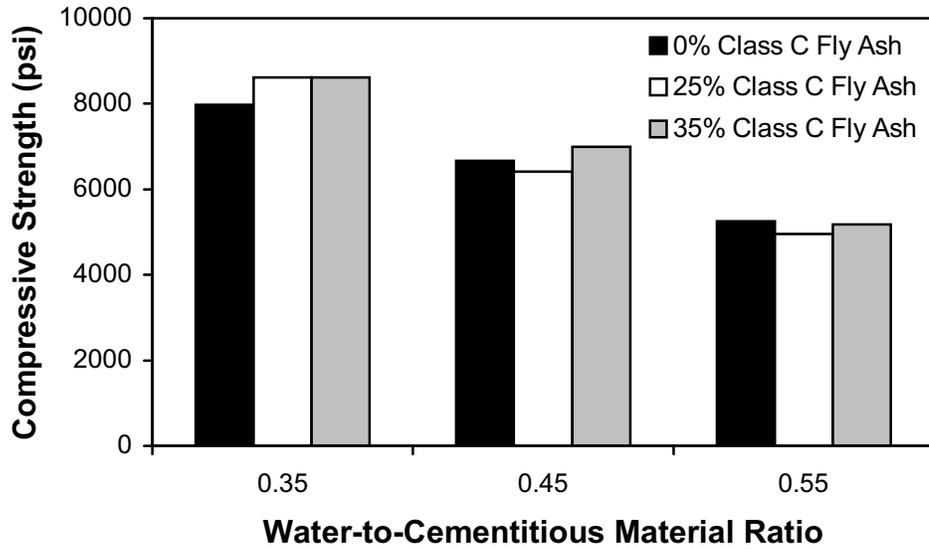


Figure 5.9 – 28-Day Room-Temperature Cured Compressive Strengths for Concretes Containing Class C Fly Ash
(Conversion: 1 psi = 0.006895 MPa)

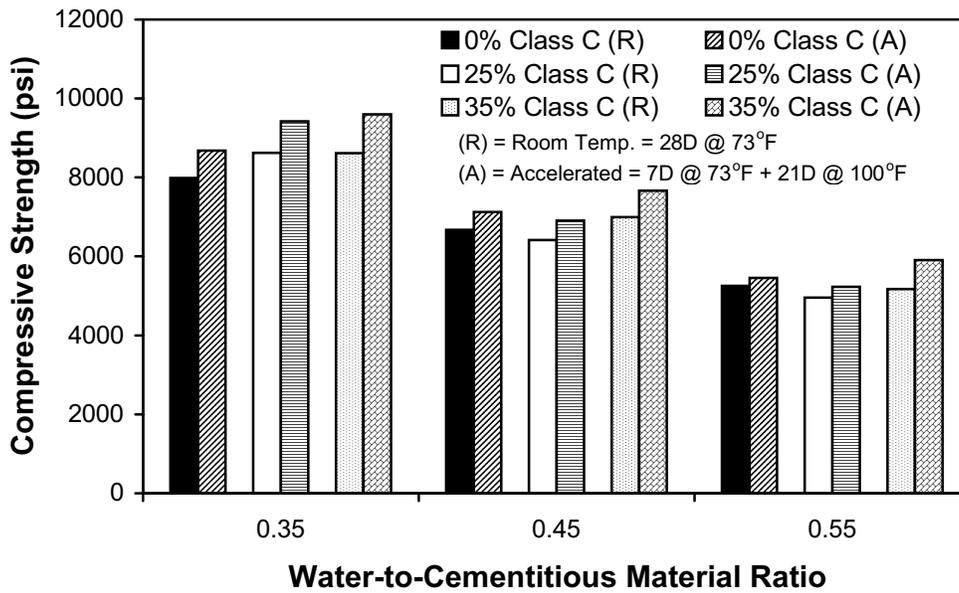


Figure 5.10 – Comparison of 28-Day Room-Temperature and Accelerated Cured Compressive Strengths for Concretes Containing Class C Fly Ash
(Conversion: 1 psi = 0.006895 MPa)

The level of strength increases obtained from accelerated curing the Class C fly ash concretes were just slightly greater than those obtained for the plain portland cement concretes. Strength increases ranged from between 5 and 14 percent, with the highest percent strength increases occurring in the concretes containing a 35 percent volumetric replacement of the fly ash. The results indicate that the Class C fly ash, like the plain portland cements, obtains most of its strength capacity within the first 28-day curing period.

Figure 5.11 displays the 28-day room-temperature cured compressive strengths of the concretes containing the Class F fly ash and the corresponding plain Type I-II (B) cement concretes. These strengths are compared with the 28-day accelerated cured strengths in Figure 5.12. For moist, room-temperature curing, the level of strength obtained in the concretes containing the Class F fly ash depended on the fly ash content. Compressive strengths decreased as the fly ash content increased. The concretes containing a 30 percent volumetric replacement of Class F fly ash had lower strengths than the plain cement concretes with the same water-to-cementitious material ratios. Concretes containing a 20 percent Class F fly ash replacement had strengths that were approximately equal to the strengths of the concretes with 25 percent Class C fly ash. These 20 percent Class F fly ash replacement concrete strengths were slightly below the plain portland cement concretes' strengths when water-to-cementitious material ratios of 0.45 and 0.55 were used while the strength was 3.4 MPa (500 psi) stronger when a 0.35 ratio was used. While past experience has shown that the use of Class F fly ashes typically yields drops in 28-day compressive strengths, it is not surprising that this Phase 2 Class F fly ash produced some strengths that were comparable if not higher than plain cement concrete strengths. The fly ash has a calcium oxide (CaO) content of 17.3 that is fairly high for a Class F fly ash. The high calcium level improves the ability of the fly ash to develop strength in concrete. However, results do show that the strength development does decrease as more cement is replaced with the Class F fly ash.

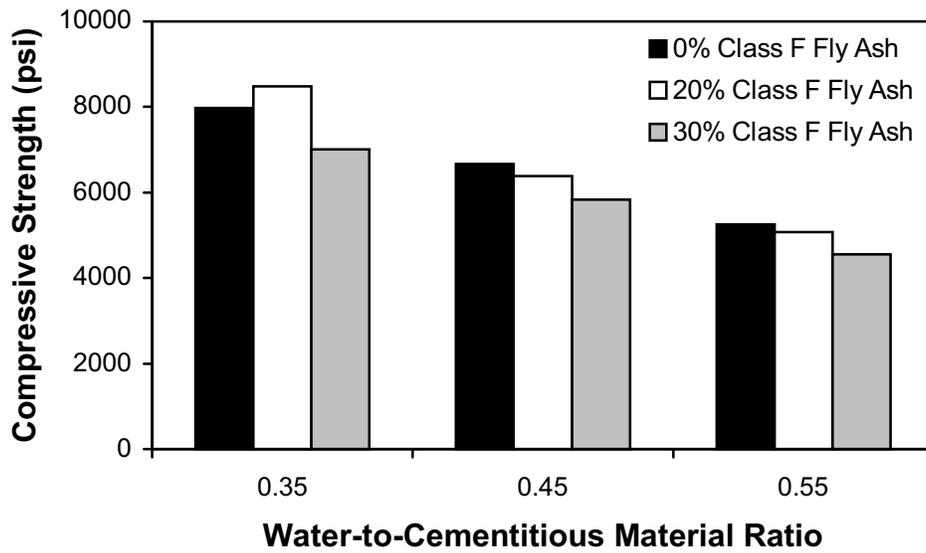


Figure 5.11 – 28-Day Room-Temperature Cured Compressive Strengths for Concretes Containing Class F Fly Ash
(Conversion: 1 psi = 0.006895 MPa)

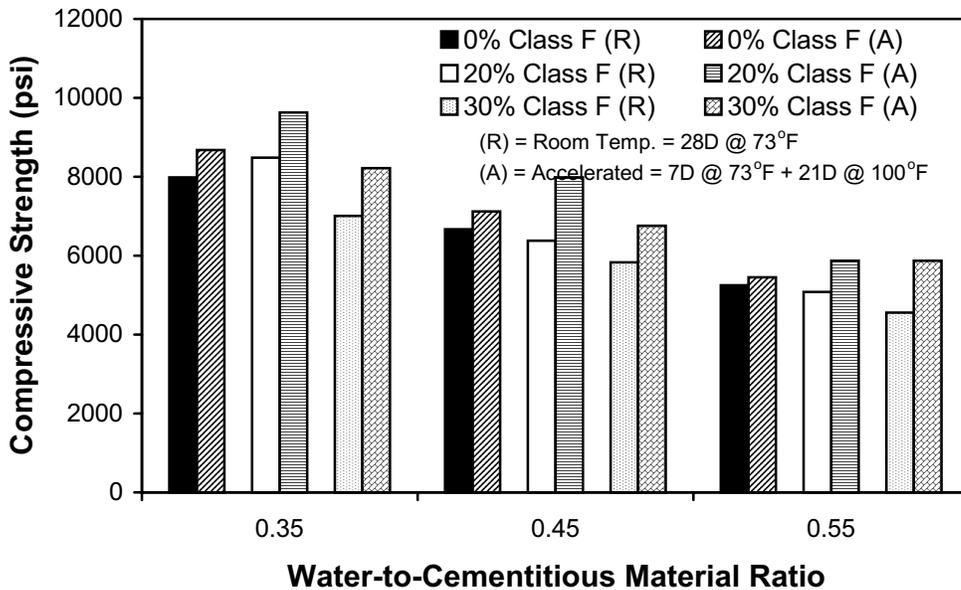


Figure 5.12 – Comparison of 28-Day Room-Temperature and Accelerated Cured Compressive Strengths for Concretes Containing Class F Fly Ash
(Conversion: 1 psi = 0.006895 MPa)

The levels of strength increase were the highest of all in the Phase 2 concretes when accelerated curing was used with concretes containing Class F fly ash. Strength increases ranged from 14 to 29 percent as accelerated curing yielded strengths as high as 11.0 MPa (1,600 psi) greater than room-temperature cured strengths. These results indicate that the Phase 2 Class F fly ash takes more time than the Phase 2 Class C fly ash and the cements to develop its full strength capacity. Some results showed that when enough heat or time is provided, the Class F fly ash can help to provide some significant concrete strength increases.

5.3.2 *Effects of Mix Properties and Curing Conditions on Permeability*

One major goal of Phase 2 testing was to demonstrate the several factors that influence the permeability of concrete. The following paragraphs discuss the effect of water-to-cementitious material ratio, cement type, mineral admixtures, and curing conditions on concrete permeability as Phase 2 28-day permeability results are discussed. A good way to qualitatively analyze the coulomb values obtained in Phase 2 testing is to use the terms established by Whiting for interpreting results obtained from the ASTM C1202 test. According to Whiting's research, coulomb values of 4,000 or higher indicate high concrete chloride penetrability, values between 2,000 and 4,000 indicate moderate penetrability, values between 1,000 and 2,000 indicate low penetrability, and values between 100 and 1,000 indicate very low penetrability.

For the moist, room-temperature cured, plain portland cement concretes, 28-day coulomb values ranged between 2,170 and 6,020 as permeability decreased with decreasing water-to-cement ratios. The results for these concretes are displayed in Figure 5.13. As expected, the 0.35 water-to-cement ratio mixes had low to moderate penetrability; the 0.45 water-to-cement ratio mixes had moderate to high penetrability; and the 0.55 water-to-cement ratio mixes had high penetrability. While the permeability results display the clear impact of the water-to-cement ratio on permeability, no trends can be observed concerning the effects of cement type on permeability. In fact, it appears, based on Phase 2 results, that cement type has little to no impact on concrete permeability.

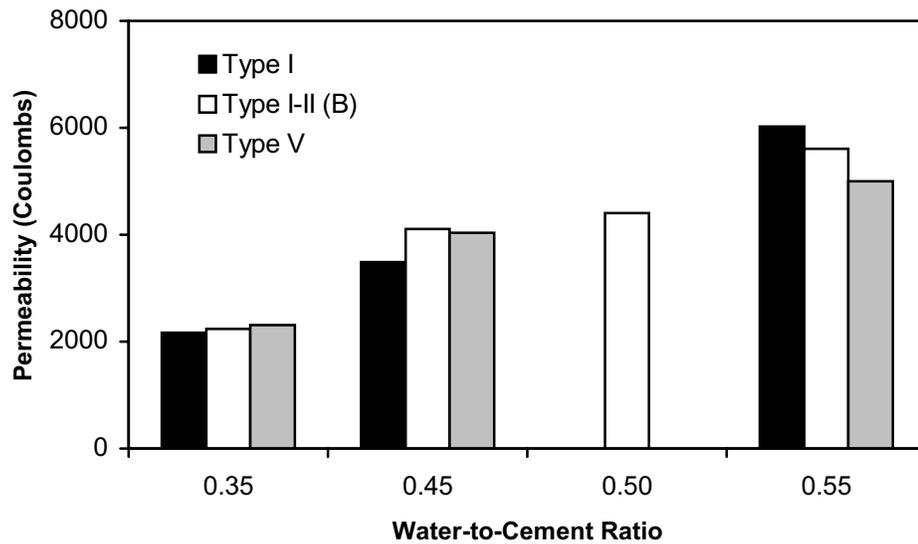


Figure 5.13 – 28-Day Room-Temperature Cured Chloride Ion Permeabilities for Plain Portland Cement Concretes

Figure 5.14 compares the 28-day permeabilities of the accelerated cured plain portland cement concretes with the room-temperature cured results. Reductions in permeability owing to accelerated curing ranged from 13 to 26 percent. Having the percent reductions in permeability owing to accelerated curing be higher than the percent increases in compressive strength indicates it may take more heat or simulated time for a plain portland cement concrete to develop its lowest permeability than it does for it to develop its ultimate compressive strength. This trend has been observed in the concrete industry for a long time. Research has shown it takes from 90 days to 1 year of room-temperature curing for a concrete to reach its ultimate level of permeability. Only approximately 28 days are needed for full compressive strength development.

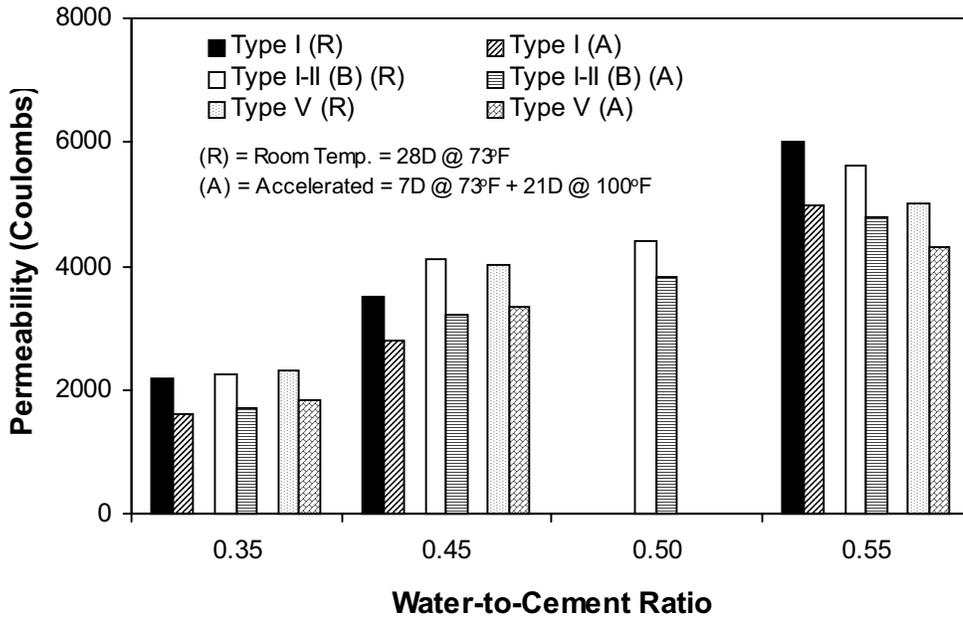


Figure 5.14 – Comparison of 28-Day Room-Temperature and Accelerated Cured Chloride Ion Permeabilities for Plain Portland Cement Concretes

For the concretes containing Class C fly ash, 28-day room-temperature cured chloride ion permeabilities were always lower than the permeability values for the corresponding plain Type I-II (B) cement concretes. Figure 5.15 shows the room-temperature cured permeabilities of the Class C fly ash concretes and the plain Type I-II (B) cement concretes. The 0.35 and 0.45 water-to-cementitious material ratio concretes with Class C fly ash had significantly lower permeabilities than the plain Type I-II (B) cement concretes with the same ratios while the concretes with the 0.55 water-to-cementitious material ratio had only slightly lower room-temperature cured permeabilities. Increases in fly ash content showed no significant affect on permeability when the Phase 2 Class C fly ash was used. These Phase 2 results correlate well with past research because past experience has supported the fact that fly ash reduces permeability. Researchers have indicated that pozzolanic reactions induced by the mineral admixtures transform large pores within cementitious systems into fine pores (Ref 12).

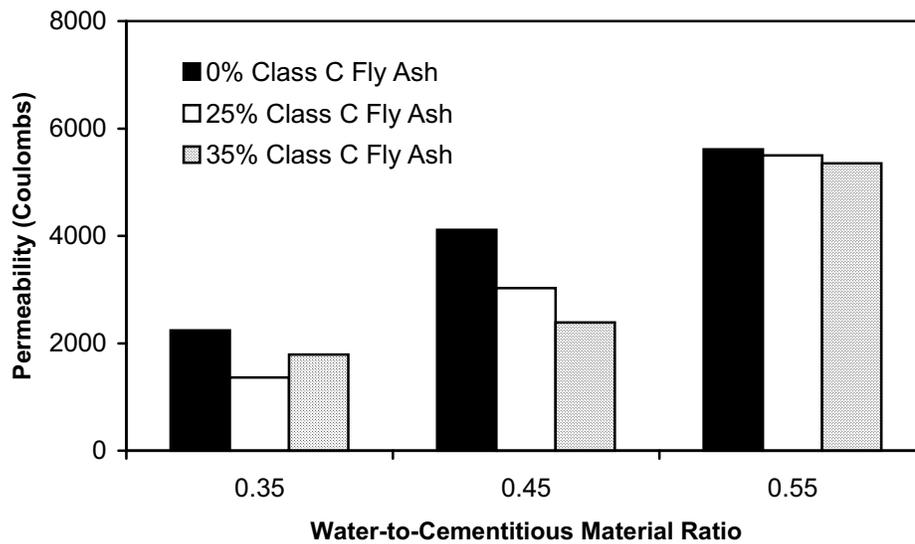


Figure 5.15 – 28-Day Room-Temperature Cured Chloride Ion Permeabilities for Concretes Containing Class C Fly Ash

Past research has also shown that concretes containing cement-mineral admixture combinations require longer curing times than do plain cement concretes to fully develop their cementitious systems and, thus, their lowest permeability. Research by Tikalsky and Carrasquillo (Ref 7) showed that after 10 days of moist curing, concretes with fly ash had comparable or higher permeabilities than equivalent plain portland cement concretes. However, after 60 and 200 days of moist curing, the permeabilities of the concretes with fly ash dropped significantly and dropped well below the permeability of the control plain portland cement concrete (Ref 7). Phase 2 results showed this same type of pattern as all the accelerated cured Class C fly ash concrete specimens had permeabilities that were at least 39 percent lower than their room-temperature cured permeabilities and were much lower than the plain portland cement concrete accelerated cured permeabilities. Accelerated and room-temperature cured permeabilities for the Class C fly ash and plain Type I-II (B) cement concretes are compared in Figure 5.16. Assuming accelerated curing simulates long-term curing, these results demonstrate that it takes time for the benefits of fly ash to truly show because fly ashes have been shown to significantly reduce concrete permeabilities with time. Knowing this property, the Virginia Department of Transportation (VDOT) proposed

specifying accelerated curing for the preparation of concrete specimens for rapid permeability testing for low-permeability specifications for transportation facilities (Ref 46). VDOT understood that the simulated long-term curing provided by accelerated curing is needed to display the full benefits of mineral admixtures such as fly ash that may be useful for application in a facility exposed to chemical attack.

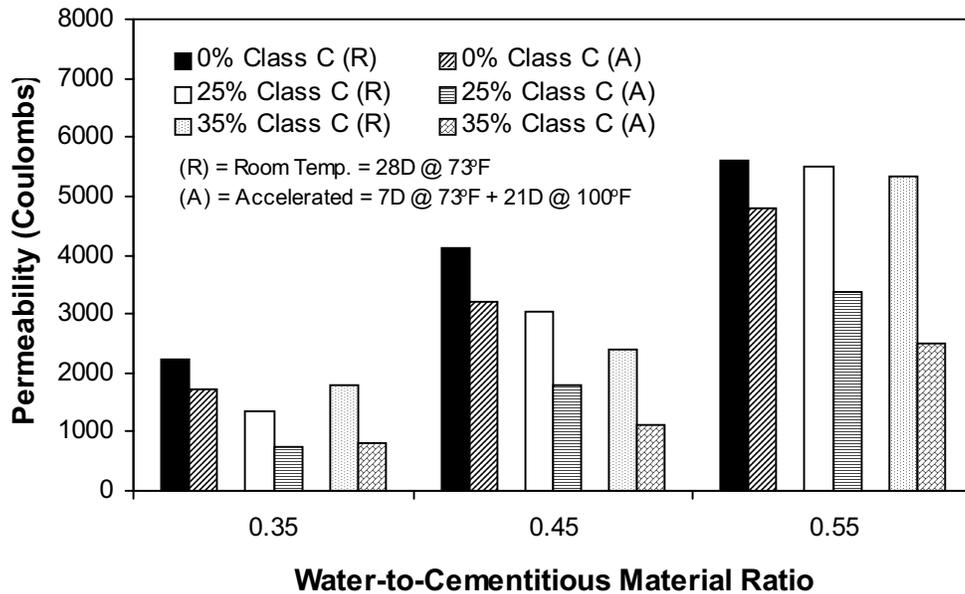


Figure 5.16 – Comparison of 28-Day Room-Temperature and Accelerated Cured Chloride Ion Permeabilities for Concretes Containing Class C Fly Ash

The benefits of fly ash concerning permeability were shown even more so in Phase 2 testing when the Class F fly ash was used. Figure 5.17 shows the 28-day room-temperature cured permeabilities of the concretes containing Class F fly ash and the corresponding plain Type I-II (B) cement concretes. Permeabilities of the concretes with the Class F fly ash were as much as 3,150 coulombs lower than the concrete without fly ash as the reductions increased with increasing fly ash content. Tikalsky and Carrasquillo also found that the Class F fly ash they tested produced lower permeabilities than the Class C fly ash. It appears low -calcium fly ashes may yield the greatest benefits as far as permeability reduction (Ref 7).

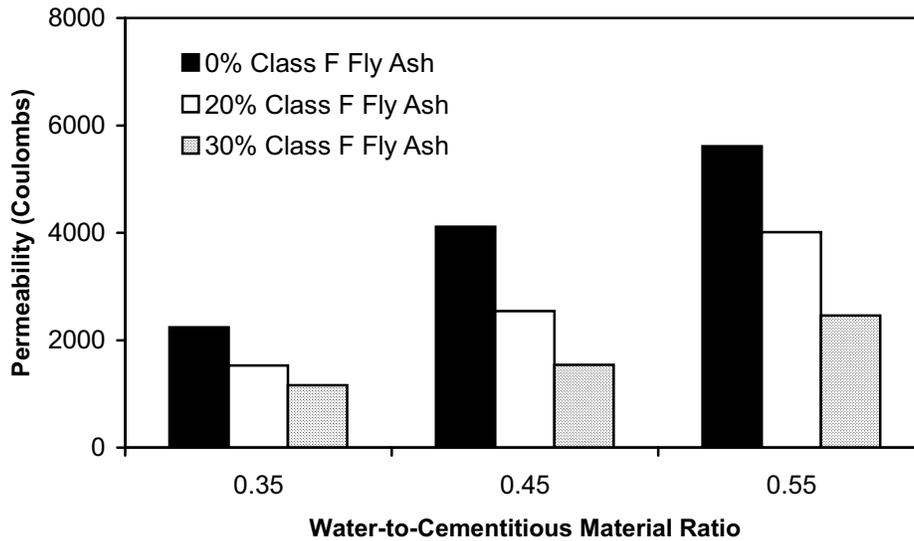


Figure 5.17 – 28-Day Room-Temperature Cured Chloride Ion Permeabilities for Concrete Containing Class F Fly Ash

Accelerated curing of the concretes with Class F fly ash yielded extremely high and consistent reductions in 28-day permeabilities as compared to the room-temperature cured results. Permeabilities were reduced between 60 and 64 percent as most 28-day accelerated cured permeabilities were less than 1,000 coulombs, with the highest value being 1,590 coulombs. Figure 5.18 compares 28-day room temperature and accelerated cured permeabilities for the Class F fly ash concretes and the plain Type I-II (B) cement concretes. These results again show that concretes with the Phase 2 Class F fly ash took the longest to develop their pore structures and cementitious systems and simulated long-term curing resulted in the largest percent strength increases and percent permeability reductions.

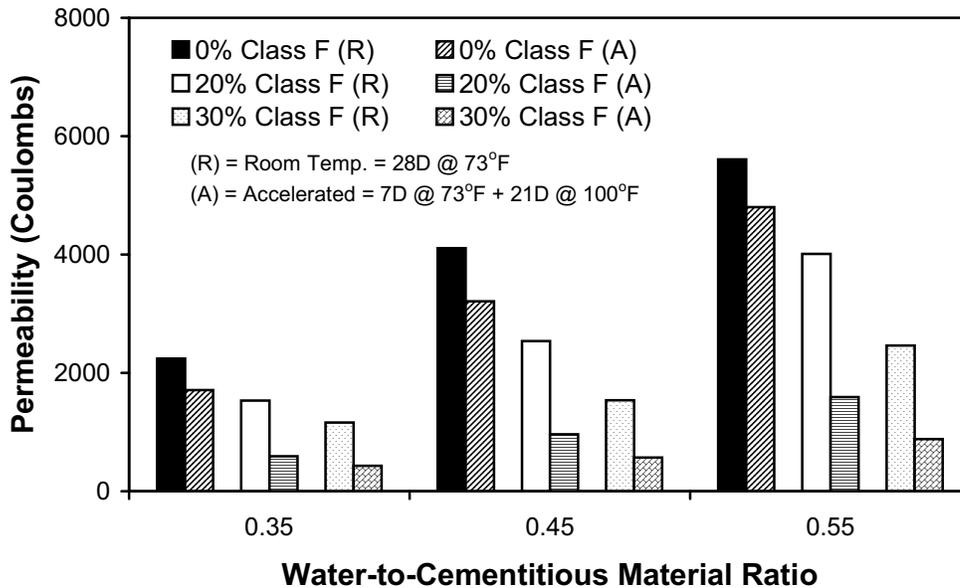


Figure 5.18 – Comparison of 28-Day Room-Temperature and Accelerated Cured Chloride Ion Permeabilities for Concretes Containing Class F Fly Ash

5.3.3 Implications of Results Concerning Specifications

While no final conclusions can be drawn concerning the impact of Phase 2 testing on the specifications for sulfate resistant concretes until long-term Phase 2 sulfate exposure test results are obtained, some trends impacting the performance-based specifications can be observed from results obtained to date. Specifically, Phase 2 results reveal problems with the water-to-cementitious material ratio limits that are currently specified for sulfate resistant concrete mix designs in current ACI and UBC guidelines and in the proposed specifications developed earlier in this project.

While there is little research showing how the current sulfate resistance specification water-to-cementitious material ratio limits were derived, it can generally be said the ratios are specified to ensure that good quality concrete is used in sulfate environments. Good quality concrete can be defined as concrete having acceptable compressive strength and, more importantly, low to moderate permeability. The problem with the water-to-cementitious material ratio specification is that it assumes that the water-to-cementitious material ratio is the only concrete property that affects permeability. Phase 2 results have demonstrated that

this is not the case. While the water-to-cementitious material ratios of Phase 2 concretes significantly impacted the concrete permeability, results showed that mineral admixtures, such as fly ash, and the curing conditions have equally important effects. These effects can be observed by looking at the concretes having a water-to-cementitious material ratio of 0.55 in Figure 5.18. While each concrete had the same water-to-cementitious material ratio, the concretes contained different levels of Class F fly ash and were cured under different conditions. The results show six different permeabilities that ranged from a high value of 5,610 coulombs to a very low value of 880 coulombs. The addition of fly ash significantly lowered permeabilities, as did curing the specimens under higher heat conditions.

Phase 2 results indicate that specifying a maximum water-to-cementitious material ratio in sulfate resistance specifications, or in any specifications for that matter, hinders engineers and limits their options for achieving good quality concrete. The specification does not allow engineers to take advantage of such new technologies as match temperature curing and fly ash, slag, and silica fume mineral admixtures. Phase 2 results proved that good quality can be achieved in a high water-to-cementitious material ratio concrete with the use of the proper amount of mineral admixtures and/or improved curing procedures. As explained in the introduction of this report, one major goal in the development of performance-based specifications is to provide engineers with the freedom to utilize any available technology to produce a concrete that meets durability needs for a given job condition. The engineer has only to show through testing that his or her concrete has the properties necessary for adequate performance. Current water-to-cementitious material ratio limits do not provide engineers this freedom.

One approach to changing the way the proposed performance-based specifications define good quality concrete, and thus acceptable compressive strengths and permeability in sulfate environments, is to make the following modifications:

1. Change water-to-cementitious material ratio limits to some form of maximum allowable 28-day coulomb values as tested by ASTM C1202.

2. Add minimum compressive strength requirements similar to those specified in ACI 318 and the UBC.

For permeability specifications, it would be effective to require test specimens be prepared following the accelerated curing procedures of moist curing 7 days at 23 °C (73 °F) and then moist curing 21 days at 38 °C (100 °F). Phase 2 results show this curing procedure allows engineers to demonstrate the long-term benefits of mineral admixtures in a short period of time. As far as the coulomb value quantities that should be specified, it is not possible to come up with values based on the limited amount of data collected in Phase 2 testing. Ideas for values can be obtained from the results for the plain Type I-II (B) cement concrete with a water-to-cementitious material ratio of 0.50 and the plain Type V cement concrete with a water-to-cementitious material ratio of 0.45 tested in Phase 2. These concretes correspond directly with the upper limit concrete types specified in current guidelines for moderate and severe sulfate environments, respectively. The Type I-II (B) cement concrete had a 28-day accelerated cured permeability of 3,820 coulombs, while the Type V cement concrete had a 28-day accelerated cured permeability of 3,350 coulombs. Simply based on these results, it could be said that concretes must have a 28-day accelerated cured permeability of 3,820 coulombs or less to be used in a moderate sulfate environment and a permeability of 3,350 coulombs or less to be used in a severe sulfate environment. Another option is to allow engineers to test their higher water-to-cementitious material ratio mix design against the equivalent plain portland cement, 0.50 water-to-cementitious material ratio concrete for moderate sulfate environments or the plain portland cement, 0.45 water-to-cementitious material ratio concrete for severe sulfate environments. If the engineer's proposed concrete permeability value is lower than the control concrete value when the accelerated curing procedures and ASTM C1202 are used, the engineer's mix design has met specifications.

Two more steps are needed to prove these maximum allowable coulomb value specifications are effective and necessary. First, results must demonstrate that concrete permeabilities affect concrete sulfate resistance more than do water-to-cementitious material

ratios. Once long-term Phase 2 sulfate exposure test results are obtained, 540-day sulfate expansions and mass changes can be compared to the permeability and the water-to-cementitious material ratios of the concretes. The results can be used to determine which property most directly impacts sulfate resistance. If results are as expected and it is indeed true that the maximum water-to-cementitious material ratios are simply specified to ensure desirable concrete permeability, the specification changes proposed in this section should be implemented. Including these changes will then ensure that the material properties that most directly impact the resistance of concrete to sulfate attack are the ones addressed in the new performance-based specifications.

The final step in evaluating permeability specifications involves establishing the level of impact of permeability on sulfate resistance and comparing this impact to the impact of cementitious material chemistry. Some past research has demonstrated that over a long period of time low permeability cannot prevent sulfate attack if the cementitious material chemistry of the concrete is vulnerable (Refs 7, 12). This problem does not impact specifications too much because no portion of the current specifications allows engineers to overcome poor cementitious chemistry with extremely low permeability. A more interesting question that Phase 2 results may answer is whether high permeability necessarily hurts concretes with severe sulfate resistant cementitious material chemistry. Perhaps a concrete with Type V cement will be resistant to sulfate attack no matter what its level of permeability. If this is true, maximum coulomb value limits for severe sulfate resistant environments may not need to be too stringent or may not be necessary at all. These issues will be addressed in future reports when all Phase 2 results have been obtained.

Chapter Six: Conclusions and Recommendations

6.1 Introduction

This research project involved an experimental investigation of the resistances of various mortars and concretes to sulfate attack for the purpose of establishing performance-based specifications for durable concrete in sulfate environments. The test program for this research was divided into two phases. Phase 1 involved evaluating a potential performance test, ASTM C1012-95, that could be used for examining the sulfate resistance of cementitious materials. Phase 2 involved evaluating how concrete mix properties and curing conditions influenced permeability and how permeability impacted concrete sulfate resistance. This chapter highlights the conclusions that were drawn from the test results obtained to date and the recommendations that were developed concerning performance-based specifications.

6.2 Conclusions

6.2.1 *Conclusions from Phase 1 Test Results*

1. Sulfate expansions of ASTM C1012-tested plain portland cement mortars confirmed the impact cement tricalcium aluminate (C_3A) contents have on sulfate resistance as increasing C_3A contents yielded increased expansions.
2. In comparing the ASTM C1012-95 expansion limits and the ASTM C150 C_3A content limits used for determining levels of sulfate resistance for portland cements, the expansion limits sometimes proved to be conservative in comparison to ASTM C150 limits. In one case, an ASTM C150 Type I-II moderate sulfate resistant cement was found to be inadequate for moderate sulfate environments according to current ASTM C1012 expansion criteria.
3. Phase 1 test results demonstrated that engineers should take caution when using borderline cements with C_3A contents between 7 and 8 percent or 4 and 5 percent in

moderate or severe sulfate environments, respectively, because the cement may be inadequate for the environment.

4. Test results for mortars containing Class F fly ash showed the fly ash significantly reduced sulfate expansions as each mortar met ASTM C1012 expansion criteria for severe sulfate resistance. These results correlate well with what was expected based on past experience.
5. Test results for mortars containing Class C fly ash showed the fly ash significantly increased expansions as each mortar proved to be inadequate for application in any sulfate environment. Results again correlated well with past experience.
6. Evaluations of mortars containing slag confirmed the expected result that slag improves sulfate resistance. Sulfate expansions were reduced enough that each mortar met ASTM C1012 expansion criteria for severe sulfate resistance.
7. An overall evaluation of ASTM C1012-95 showed the test produced consistent and reliable data for any mortar with expansions in the severe and moderate sulfate resistance range. Test results correlated well with past experience as the test accurately revealed the effects of various portland cements and cement-mineral admixture combinations on sulfate resistance. Expansion performance criteria corresponding to ASTM C1012 proved to be acceptable as the limits were shown to at least be conservative and thus safe.

6.2.2 Conclusions from Phase 2 Test Results

1. The 28-day room-temperature cured specimen compressive strengths for plain portland cement concretes revealed that the strengths decreased when the water-to-cement ratio increased. Cement type had little impact on the level of strength.
2. The 28-day accelerated cured strengths were only slightly higher than the room-temperature cured strengths for the plain cement concretes. The increased heat did little to improve strength because the concretes developed most of their strength capacity within the first 28 days of room-temperature curing.

3. The 28-day room-temperature cured specimen compressive strengths for the concretes with Class C fly ash were similar to plain cement concrete strengths in that the fly ash did little to reduce compressive strength and actually increased strengths when the 0.35 water-to-cementitious material ratio was used.
4. Accelerated curing of the concretes with Class C fly ash increased strengths slightly more than in plain portland cement concretes. However, it appeared the Class C fly ash concretes had also gained a majority of their strength within the first 28 days of room-temperature curing.
5. The 28-day room-temperature cured compressive strengths for the concretes with Class F fly ash depended on the fly ash content. The lower fly ash content of 20 percent volumetric replacement produced strengths similar to the plain portland cement concrete strengths while the higher 30 percent content yielded concretes with lower compressive strengths.
6. The highest strength improvements owing to accelerated curing occurred in the Class F fly ash concretes as strengths for both the low and high fly ash content concretes were nearly equivalent to or higher than plain portland cement concrete strengths after accelerated curing.
7. The 28-day room-temperature cured specimen chloride ion permeabilities for the plain portland cement concretes increased as the water-to-cement ratio increased. The cement type had little impact on coulomb values.
8. The 28-day accelerated cured plain portland cement concrete permeabilities were slightly lower than the room-temperature cured values. When accelerated curing procedures were used, percent reductions in permeability were higher than reductions in compressive strength for the plain portland cement concrete specimens.
9. The 28-day room-temperature cured specimen permeabilities for the concretes with Class C fly ash were always lower than plain portland cement concrete values.

10. Reductions in permeability owing to accelerated curing for the concretes with Class C fly ash were much higher than reductions in the plain portland cement concretes. Results indicate the fly ash requires more time to develop the full cement matrix and thus the optimum concrete permeability.
11. The 28-day room-temperature cured specimen permeabilities for the concretes with Class F fly ash were the lowest of all the concretes. The permeability dropped with increasing fly ash content.
12. Accelerated curing of the concretes with Class F fly ash resulted in the largest reductions in permeability. Permeabilities were as much as 3,920 coulombs lower than the plain portland cement concrete values.

6.3 Recommendations

6.3.1 Recommendations Based on Phase 1 Results

1. Phase 1 results indicate ASTM C1012-95 produces reliable results that correlate well with past laboratory experience. ASTM C1012-95 has proven to be an adequate performance test that can be used in performance-based specifications to evaluate the sulfate resistance of portland cements and cement–mineral admixture combinations.
2. The mortar bar expansion performance criteria corresponding to ASTM C1012 have proven to be a fairly accurate predictor of portland cement and cement–mineral admixture combination sulfate resistance levels. No changes are recommended concerning the expansion criteria because they were shown to be conservative and thus safe.
3. The elimination of the mass change criteria proposed in an earlier report for this TxDOT project has been recommended. The mass change criteria corresponding to ASTM C1012 proved to be ineffective because the measurement is difficult to perform without

interfering with expansion results and the criteria are evaluating a damage mechanism, surface scaling, that will most likely not occur in ASTM C1012 mortar bar specimens.

4. An addition to the performance-based specifications has been proposed concerning borderline cements with C_3A contents between 7 and 8 percent and 4 and 5 percent. The recommendation involves requiring engineers to test these borderline cements using ASTM C1012 if the cements are to be used in moderate or severe sulfate environments, respectively. Engineers must prove these cements meet ASTM C1012 performance criteria requirements for their service environment, unless further research proves that these cements are adequate for use in moderate and severe sulfate environments.

6.3.2 *Recommendations Based on Phase 2 Results*

1. Based on Phase 2 results that clearly showed that the water-to-cementitious material ratio of a concrete mix was not the only factor influencing concrete permeability, it has been recommended that the water-to-cementitious material ratio limits be replaced by a proven and reliable performance-based test indicator of the permeability characteristics of the concrete. Such alternatives as using coulomb value limits for different sulfate environments seem promising but further research is needed prior to its full implementation.
2. Final coulomb value limits cannot be established until long-term, Phase 2 sulfate exposure test results are obtained. Long-term results will determine if it is permeability and not water-to-cementitious material ratio values that most directly impacts sulfate resistance. Also, the results will reveal the impact of permeability on sulfate resistance compared to the impact of cementitious material chemistry. This information can be used to determine the necessity of having concrete permeability requirements for each sulfate environment.

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Appendix A: Mix Proportions for Phase 1 Mortars

Appendix A: Mix Proportions for Phase 1 Mortars

Table A.1 – Mix Proportions for Phase 1 Mortars

Batch Name	Cement Type	Fly Ash/Slag (% Rep.)*	Water (lb/yd ³)	Portland Cement (lb/yd ³)	Fly Ash/Slag (lb/yd ³)	Graded Sand – dry (lb/yd ³)
PC1	I-II (A)	None	447	897	0	2,455
1FC-1	I-II (A)	C (25)	430	673	189	2,455
1FC-2	I-II (A)	C (35)	424	583	264	2,455
1FA-1	I-II (A)	F (20)	441	718	144	2,455
1FA-2	I-II (A)	F (30)	435	628	216	2,455
S1	I-II (A)	Slag (50)	468	449	407	2,455
PC2	I-II (B)	None	447	897	0	2,455
2FC-1	I-II (B)	C (25)	431	673	189	2,455
2FC-2	I-II (B)	C (35)	423	583	264	2,455
2FA-1	I-II (B)	F (20)	440	718	144	2,455
2FA-2	I-II (B)	F (30)	435	628	216	2,455
S2	I-II (B)	Slag (50)	470	449	407	2,455
PC5	V	None	447	897	0	2,455
5FC-1	V	C (25)	432	673	189	2,455
5FC-2	V	C (35)	426	583	264	2,455
5FA-1	V	F (20)	443	718	144	2,455
5FA-2	V	F (30)	438	628	216	2,455
S5	V	Slag (50)	480	449	407	2,455
P1	I	None	447	897	0	2,455
1C-1	I	C (25)	430	673	189	2,455
1C-2	I	C (35)	422	583	264	2,455
1F-1	I	F (20)	439	718	144	2,455
1F-2	I	F (30)	434	628	216	2,455
Slag-1	I	Slag (50)	462	449	407	2,455
* In this column, C stands for ASTM Class C fly ash and F stands for ASTM Class F fly ash.						
** Conversion: 1 lb/yd ³ = 0.5933 kg/m ³						

Appendix B: Mix Proportions for Phase 2 Concretes

Appendix B: Mix Proportions for Phase 2 Concretes

Table B.1 – Mix Proportions for Phase 2 Concrete Batches

Batch Details				Aggregate Moisture Contents		Mix Proportions (fl oz/yd ³ for HRWR and lb/yd ³ for others)					
Batch Name	Cement Type	Fly Ash* (% Rep.)	W/C (by weight)	CA (%)	FA (%)	HRWR	Water	Portland Cement	Fly Ash	FA	CA
PC1-1	I	None	0.35	0.34	1.53	82	193	564	0	1441	1786
PC1-2	I	None	0.45	1.26	3.37	41	212	564	0	1408	1802
PC1-3	I	None	0.55	1.26	3.37	0	275	564	0	1350	1802
PC2-1	I-II (B)	None	0.35	0.34	1.53	55	195	564	0	1441	1786
PC2-2	I-II (B)	None	0.45	0.42	1.88	9	249	564	0	1388	1787
PC2-3	I-II (B)	None	0.50	0.74	2.43	0	266	564	0	1367	1793
PC2-4	I-II (B)	None	0.55	0.74	2.43	0	296	564	0	1338	1793
PC5-1	V	None	0.35	0.32	0.90	100	202	564	0	1432	1786
PC5-2	V	None	0.45	0.28	1.56	27	255	564	0	1384	1785
PC5-3	V	None	0.55	0.50	1.15	0	315	564	0	1321	1789
2FC-1a	I-II (B)	C (25)	0.35	0.34	1.91	68	180	423	119	1477	1786
2FC-2a	I-II (B)	C (25)	0.45	0.35	1.84	5	241	423	119	1421	1786
2FC-3a	I-II (B)	C (25)	0.55	0.35	1.84	0	297	423	119	1365	1786
2FC-1b	I-II (B)	C (35)	0.35	0.26	1.17	68	190	367	166	1478	1785
2FC-2b	I-II (B)	C (35)	0.45	0.24	0.95	5	251	367	166	1421	1784
2FC-3b	I-II (B)	C (35)	0.55	0.24	0.95	0	305	367	166	1367	1784
2FA-1a	I-II (B)	F (20)	0.35	0.28	1.34	82	189	451	91	1468	1785
2FA-2a	I-II (B)	F (20)	0.45	0.27	1.20	9	251	451	91	1411	1785
2FA-3a	I-II (B)	F (20)	0.55	0.27	1.20	0	306	451	91	1357	1785
2FA-1b	I-II (B)	F (30)	0.35	0.18	1.57	103	182	395	136	1487	1783
2FA-2b	I-II (B)	F (30)	0.45	0.26	0.82	9	251	395	136	1422	1785
2FA-3b	I-II (B)	F (30)	0.55	0.25	2.11	0	289	395	136	1386	1784

* In this column, C stands for ASTM Class C fly ash and F stands for ASTM Class F fly ash
 ** Conversion: 1 lb/yd³ = 0.5993 kg/m³

Appendix C: Compressive Strength Results for Phase 1 Mortars

Appendix C: Compressive Strength Results for Phase 1 Mortars

Table C.1 – Compressive Strengths of Mortars with Type I-II (A) Cement

Batch Name	PC1	1FC-1	1FC-2	1FA-1	1FA-2	S1
Fly Ash/Slag (% Rep.)	None	Class C (25%)	Class C (35%)	Class F (20%)	Class F (30%)	Slag (50%)
Age (days)*	Compressive Strengths (psi)**					
1	2660	1930	1810	2070	1980	1310
2	3250	2380	2230	2340	2210	2030
3	--	2780	2300	2510	2540	2670
4	--	3040	2700	2600	2770	2900
5	--	--	2820	3210	2930	--
6	--	--	2980	--	--	--
* Time of measurements rounded to nearest day. Strength after 1 day is after 24 hours of accelerated curing at 95°F. Moist cured in limewater at 73 °F for remainder of time.						
** Conversion: 1 psi = 0.006895 MPa						

Table C.2 – Compressive Strengths of Mortars with Type I-II (B) Cement

Batch Name	PC2	2FC-1	2FC-2	2FA-1	2FA-2	S2
Fly Ash/Slag (% Rep.)	None	Class C (25%)	Class C (35%)	Class F (20%)	Class F (30%)	Slag (50%)
Age (days)*	Compressive Strengths (psi)**					
1	2800	2080	1760	1870	1860	1600
2	3610	2430	2200	2250	2300	2180
3	--	2770	2420	2630	2500	2280
4	--	3010	2730	2700	2830	2980
5	--	--	--	--	3020	3080
6	--	--	--	3040	--	--
7	--	--	3380	--	--	--
* Time of measurements rounded to nearest day. Strength after 1 day is after 24 hours of accelerated curing at 95°F. Moist cured in limewater at 73 °F for remainder of time.						
** Conversion: 1 psi = 0.006895 MPa						

Table C.3 – Compressive Strengths of Mortars with Type V Cement

Batch Name	PC5	5FC-1	5FC-2	5FA-1	5FA-2	S5
Fly Ash/Slag (% Rep.)	None	Class C (25%)	Class C (35%)	Class F (20%)	Class F (30%)	Slag (50%)
Age (days)*	Compressive Strengths (psi)**					
1	2050	1280	1290	1510	1100	1030
2	2610	2020	1630	1560	1550	--
3	2700	2100	2020	2020	1820	2090
4	3190	2520	2110	2250	--	--
5	--	2800	--	--	1920	2600
6	--	3110	2590	--	2160	--
7	--	--	2940	2950	2490	3100
8	--	--	--	--	2540	--
9	--	--	--	--	2600	--
12	--	--	--	--	2860	--
* Time of measurements rounded to nearest day. Strength after 1 day is after 24 hours of accelerated curing at 95°F. Moist cured in limewater at 73 °F for remainder of time.						
** Conversion: 1 psi = 0.006895 MPa						

Table C.4 – Compressive Strengths of Mortars with Type I Cement

Batch Name	P1	1C-1	1C-2	1F-1	1F-2	Slag-1
Fly Ash/Slag (% Rep.)	None	Class C (25%)	Class C (35%)	Class F (20%)	Class F (30%)	Slag (50%)
Age (days)*	Compressive Strengths (psi)**					
1	2500	2230	1940	2150	1720	1610
2	2950	--	2360	2540	2130	1990
3	--	2670	2670	--	--	--
4	--	2800	2810	2760	2440	2510
5	--	3240	3220	3100	2650	2670
6	--	--	--	--	2800	2820
7	--	--	--	--	2950	2920
*Time of measurements rounded to nearest day. Strength after 1 day is after 24 hours of accelerated curing at 95°F. Moist cured in limewater at 73 °F for remainder of time.						
** Conversion: 1 psi = 0.006895 MPa						

Appendix D: Sulfate Expansion Results for Phase 1 Mortars

Appendix D: Sulfate Expansion Results for Phase 1 Mortars

Table D.1 – Sulfate Expansions of Mortars with Type I-II (A) Cement

Batch Name	PC1			1FC-1			1FC-2		
Fly Ash/Slag (% Rep.)	None			Class C (25%)			Class C (35%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.004	6	0.001	0.006	5	0.001	0.010	6	0.001
14	0.008	6	0.001	0.011	5	0.000	0.018	6	0.002
21	0.010	6	0.001	0.017	5	0.001	0.023	6	0.002
28	0.012	6	0.001	0.020	5	0.001	0.031	6	0.006
56	0.017	6	0.001	0.035	5	0.003	0.113	6	0.043
91	0.025	6	0.001	0.114	5	0.012	0.277	4	0.051
105	0.028	6	0.002	0.176	5	0.020	--	--	--
120	0.034	6	0.002	0.275	5	0.035	--	--	--
180	0.060	6	0.004	0.711	4	0.121	--	--	--
Batch Name	1FA-1			1FA-2			S1		
Fly Ash/Slag (% Rep.)	Class F (20%)			Class F (30%)			Slag (50%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.005	6	0.000	0.007	6	0.001	0.009	6	0.001
14	0.010	6	0.000	0.010	6	0.001	0.014	6	0.001
21	0.012	6	0.000	0.015	6	0.001	0.015	6	0.001
28	0.014	6	0.001	0.017	6	0.001	0.018	6	0.002
56	0.019	6	0.001	0.023	6	0.002	0.023	5	0.002
91	0.022	6	0.001	0.028	6	0.002	0.028	5	0.001
105	0.023	6	0.001	0.032	6	0.001	0.031	5	0.001
120	0.026	6	0.001	0.034	6	0.002	0.035	5	0.001
180	0.038	6	0.003	0.044	6	0.003	0.040	5	0.002

Table D.2 – Sulfate Expansions of Mortars with Type I-II (B) Cement

Batch Name	PC2			2FC-1			2FC-2		
Fly Ash/Slag (% Rep.)	None			Class C (25%)			Class C (35%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.003	6	0.001	0.005	6	0.000	0.007	6	0.001
14	0.006	6	0.001	0.010	6	0.000	0.013	6	0.001
21	0.010	6	0.001	0.015	6	0.001	0.016	6	0.001
28	0.011	6	0.001	0.018	6	0.000	0.023	6	0.002
56	0.017	6	0.001	0.031	6	0.003	0.188	6	0.045
91	0.027	6	0.001	0.109	6	0.013	--	--	--
105	0.033	6	0.001	0.189	6	0.019	--	--	--
120	0.043	6	0.001	0.305	6	0.031	--	--	--
180	0.113	6	0.008	0.974	6	0.219	--	--	--
Batch Name	2FA-1			2FA-2			S2		
Fly Ash/Slag (% Rep.)	Class F (20%)			Class F (30%)			Slag (50%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.004	6	0.001	0.005	6	0.000	0.008	6	0.001
14	0.008	6	0.001	0.008	6	0.000	0.009	6	0.001
21	0.009	6	0.001	0.010	6	0.000	0.012	6	0.001
28	0.012	6	0.001	0.013	6	0.000	0.015	6	0.001
56	0.017	6	0.001	0.017	6	0.001	0.020	6	0.002
91	0.020	6	0.001	0.020	6	0.001	0.025	6	0.002
105	0.021	6	0.001	0.022	6	0.001	0.030	6	0.002
120	0.023	6	0.002	0.022	6	0.001	0.033	6	0.002
180	0.041	6	0.004	0.026	6	0.001	0.049	6	0.007

Table D.3 – Sulfate Expansions of Mortars with Type V Cement

Batch Name	PC5			5FC-1			5FC-2		
Fly Ash/Slag (% Rep.)	None			Class C (25%)			Class C (35%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.002	6	0.001	0.007	5	0.001	0.007	6	0.000
14	0.006	6	0.000	0.010	5	0.000	0.014	6	0.001
21	0.008	6	0.001	0.014	5	0.000	0.019	6	0.001
28	0.009	6	0.001	0.017	5	0.001	0.023	6	0.001
56	0.013	6	0.000	0.027	5	0.003	0.056	6	0.004
91	0.018	6	0.000	0.054	5	0.011	0.130	6	0.010
105	0.022	6	0.000	0.069	5	0.015	0.172	6	0.012
120	0.026	6	0.001	0.096	5	0.020	0.245	6	0.017
180	0.037	6	0.001	0.185	5	0.039	0.459	6	0.034
Batch Name	5FA-1			5FA-2			S5		
Fly Ash/Slag (% Rep.)	Class F (20%)			Class F (30%)			Slag (50%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.002	5	0.001	0.003	6	0.000	0.007	5	0.000
14	0.005	5	0.000	0.007	6	0.001	0.011	5	0.001
21	0.007	5	0.001	0.009	6	0.001	0.013	5	0.001
28	0.009	5	0.000	0.011	6	0.001	0.016	5	0.001
56	0.012	5	0.000	0.017	6	0.001	0.021	5	0.001
91	0.019	5	0.001	0.021	6	0.002	0.024	5	0.001
105	0.019	5	0.001	0.023	6	0.001	0.027	5	0.001
120	0.023	5	0.001	0.026	6	0.001	0.031	5	0.001
180	0.030	5	0.001	0.030	6	0.002	0.036	5	0.001

Table D.4 – Sulfate Expansions of Mortars with Type I Cement

Batch Name	P1			1C-1			1C-2		
Fly Ash/Slag (% Rep.)	None			Class C (25%)			Class C (35%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.007	6	0.000	0.008	6	0.001	0.011	4	0.001
14	0.011	6	0.001	0.013	6	0.000	0.017	4	0.001
21	0.013	6	0.001	0.018	6	0.001	0.024	4	0.001
28	0.017	6	0.000	0.021	6	0.001	0.035	4	0.004
56	0.019	6	0.001	0.098	6	0.006	0.653	1	0.000
91	0.032	6	0.001	--	--	--	--	--	--
105	0.037	6	0.004	--	--	--	--	--	--
120	0.074	6	0.009	--	--	--	--	--	--
180	0.199	6	0.018	--	--	--	--	--	--
Batch Name	1F-1			1F-2			Slag-1		
Fly Ash/Slag (% Rep.)	Class F (20%)			Class F (30%)			Slag (50%)		
Soaking Time (Days)	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation	Avg. Expansion (%)	# of Bars	Standard Deviation
7	0.004	6	0.001	0.003	5	0.001	0.004	6	0.000
14	0.008	6	0.001	0.006	5	0.000	0.008	6	0.001
21	0.010	6	0.001	0.010	5	0.000	0.012	6	0.001
28	0.012	6	0.001	0.011	5	0.000	0.012	6	0.001
56	0.017	6	0.001	0.022	5	0.001	0.017	6	0.002
91	0.020	6	0.001	0.029	5	0.002	0.023	6	0.002
105	0.024	6	0.001	0.030	5	0.001	0.023	6	0.002
120	0.024	6	0.001	0.037	5	0.001	0.033	6	0.001
180	0.028	6	0.001	0.045	5	0.001	0.038	6	0.002

Appendix E: Compressive Strength Results for Phase 2 Concretes

Appendix E: Compressive Strength Results for Phase 2 Concretes

Table E.1 - Compressive Strengths of Plain Portland Cement Concrete

Batch Name	PC1-1	PC1-2	PC1-3	PC2-1	PC2-2
Cement Type	I	I	I	I-II (B)	I-II (B)
W/C by weight	0.35	0.45	0.55	0.35	0.45
Age (days)*	Compressive Strength (psi)**				
1	4110	--	--	4820	--
2	--	4230	2270	--	3740
3	--	--	--	--	--
7	--	--	3400	--	--
8	--	--	3620	--	--
28 (Room)	6960	6320	5150	7980	6670
28 (Accel.)	7440	6620	5490	8680	7120
Batch Name	PC2-3	PC2-4	PC5-1	PC5-2	PC5-3
Cement Type	I-II (B)	I-II (B)	V	V	V
W/C by weight	0.50	0.55	0.35	0.45	0.55
Age (days)*	Compressive Strength (psi)**				
1	--	--	2300	--	--
2	3300	2500	--	2140	1390
3	3610	--	--	--	--
4	--	--	5420	--	--
5	--	3400	--	3610	--
6	--	3570	--	--	--
7	--	--	--	--	2340
14	--	--	--	--	3190
18	--	--	--	--	3230
28 (Room)	5990	5250	8170	5620	4020
28 (Accel.)	6060	5450	8440	5470	4000
<p>* Time of measurements rounded to nearest day. Twenty-eight-day compressive strengths determined using 4-in. x 8-in. cylinders with all other strengths determined using 3-in. x 6-in. cylinders.</p> <p>** Conversion: 1 psi = 0.006895 MPa</p>					

Table E.2 – Compressive Strengths of Concretes with Fly Ash

Batch Name	2FC-1a	2FC-2a	2FC-3a	2FC-1b	2FC-2b	2FC-3b
Fly Ash (% Rep.)	Class C (25%)	Class C (25%)	Class C (25%)	Class C (35%)	Class C (35%)	Class C (35%)
W/C by weight	0.35	0.45	0.55	0.35	0.45	0.55
Age (days)*	Compressive Strengths (psi)**					
1	3950	--	--	2470	--	--
2	--	3340	2190	4680	3010	1700
3	--	4020	--	--	--	--
4	--	--	--	--	4060	--
5	--	--	3380	--	--	--
8	--	--	3690	--	--	3430
9	--	--	--	--	--	3660
28 (Room)	8620	6410	4960	8610	6990	5180
28 (Accel.)	9420	6900	5230	9600	7660	5900
Batch Name	2FA-1a	2FA-2a	2FA-3a	2FA-1b	2FA-2b	2FA-3b
Fly Ash (% Rep.)	Class F (20%)	Class F (20%)	Class F (20%)	Class F (30%)	Class F (30%)	Class F (30%)
W/C by weight	0.35	0.45	0.55	0.35	0.45	0.55
Age (days)*	Compressive Strengths (psi)**					
1	3140	--	--	1740	--	--
2	4850	3310	1950	3580	2870	1940
3	--	4100	--	--	3300	--
5	--	--	2690	--	3830	--
8	--	--	3140	--	--	--
9	--	--	--	--	--	3410
10	--	--	3470	--	--	--
12	--	--	--	--	--	3700
15	--	--	3620	--	--	--
28 (Room)	8480	6380	5080	7010	5830	4560
28 (Accel.)	9630	7980	5870	8220	6760	5870
* Time of measurements rounded to nearest day. Twenty-eight-day compressive strengths determined using 4-in. x 8-in. cylinders with all other strengths determined using 3-in. x 6-in. cylinders.						
** Conversion: 1 psi = 0.006895 MPa						

**Appendix F: Initial Sulfate Expansion and Mass Change Results
for Phase 2 Concretes**

Table F.1 - Sulfate Expansions and Mass Changes for Batch PC1-1

Batch Name	PC1-1					
Description	Type I Cement + No fly ash (w/c = 0.35)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	-0.002	0.212	0.002	0.0945	0.000	0.0687
60	0.000	0.418	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.2 - Sulfate Expansions and Mass Changes for Batch PC1-2

Batch Name	PC1-2					
Description	Type I Cement + No fly ash (w/c = 0.45)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.003	0.203	0.003	0.161	0.002	0.155
60	0.006	0.375	0.003	0.302	0.002	0.321
90	0.004	0.496	0.000	0.435	0.003	0.439
120	0.005	0.587	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.3 - Sulfate Expansions and Mass Changes for Batch PC1-3

Batch Name	PC1-3					
Description	Type I Cement + No fly ash (w/c = 0.55)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.006	0.154	0.004	0.160	0.008	0.166
60	0.010	0.273	0.005	0.348	0.013	0.349
90	0.007	0.381	0.007	0.524	0.014	0.562
120	0.020	0.533	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.4 - Sulfate Expansions and Mass Changes for Batch PC2-1

Batch Name	PC2-1					
Description	Type I-II (B) Cement + No fly ash (w/c = 0.35)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	-0.002	0.198	0.001	0.088	0.001	0.115
60	0.000	0.319	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.5 - Sulfate Expansions and Mass Changes for Batch PC2-2

Batch Name	PC2-2					
Description	Type I-II (B) Cement + No fly ash (w/c = 0.45)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.001	0.178	0.007	0.172	0.004	0.153
60	0.004	0.304	--	--	--	--
90	0.004	0.396	0.007	0.392	0.005	0.387
120	0.005	0.428	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.6 - Sulfate Expansions and Mass Changes for Batch PC2-3

Batch Name	PC2-3					
Description	Type I-II (B) Cement + No fly ash (w/c = 0.50)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.004	0.205	--	--	--	--
60	0.008	0.334	--	--	--	--
90	0.005	0.431	0.004	0.354	0.004	0.326
120	0.010	0.487	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.7 - Sulfate Expansions and Mass Changes for Batch PC2-4

Batch Name	PC2-4					
Description	Type I-II (B) Cement + No fly ash (w/c = 0.55)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.003	0.157	--	--	--	--
60	0.000	0.267	--	--	--	--
90	0.003	0.386	0.003	0.387	0.005	0.345
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.8 - Sulfate Expansions and Mass Changes for Batch PC5-1

Batch Name	PC5-1					
Description	Type V Cement + No fly ash (w/c = 0.35)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.000	0.074	0.000	0.055	0.000	0.051
60	0.001	0.197	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.9 - Sulfate Expansions and Mass Changes for Batch PC5-2

Batch Name	PC5-2					
Description	Type V Cement + No fly ash (w/c = 0.45)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.000	0.173	0.001	0.147	-0.002	0.129
60	0.000	0.262	0.000	0.243	-0.005	0.209
90	0.002	0.286	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.10 - Sulfate Expansions and Mass Changes for Batch PC5-3

Batch Name	PC5-3					
Description	Type V Cement + No fly ash (w/c = 0.55)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.001	0.179	-0.001	0.177	0.007	0.181
60	0.006	0.265	-0.001	0.208	0.009	0.223
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.11 - Sulfate Expansions and Mass Changes for Batch 2FC-1a

Batch Name	2FC-1a					
Description	Type I-II (B) Cement + 25% Class C fly ash (w/c = 0.35)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	--	--	-0.001	0.067	0.000	0.036
60	0.001	0.274	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.12 - Sulfate Expansions and Mass Changes for Batch 2FC-2a

Batch Name	2FC-2a					
Description	Type I-II (B) Cement + 25% Class C fly ash (w/c = 0.45)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	--	--	-0.003	0.098	-0.001	0.060
60	0.003	0.263	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.13 - Sulfate Expansions and Mass Changes for Batch 2FC-3a

Batch Name	2FC-3a					
Description	Type I-II (B) Cement + 25% Class C fly ash (w/c = 0.55)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.012	0.309	-0.004	0.174	-0.005	0.144
60	--	--	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.14 - Sulfate Expansions and Mass Changes for Batch 2FC-1b

Batch Name	2FC-1b					
Description	Type I-II (B) Cement + 35% Class C fly ash (w/c = 0.35)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	-0.001	0.167	0.003	0.045	0.005	0.014
60	0.003	0.235	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.15 - Sulfate Expansions and Mass Changes for Batch 2FC-2b

Batch Name	2FC-2b					
Description	Type I-II (B) Cement + 35% Class C fly ash (w/c = 0.45)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.000	0.161	0.003	0.085	0.002	0.030
60	--	--	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.16 - Sulfate Expansions and Mass Changes for Batch 2FC-3b

Batch Name	2FC-3b					
Description	Type I-II (B) Cement + 35% Class C fly ash (w/c = 0.55)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.001	0.272	0.002	0.158	0.002	0.128
60	--	--	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.17 - Sulfate Expansions and Mass Changes for Batch 2FA-1a

Batch Name	2FA-1a					
Description	Type I-II (B) Cement + 20% Class F fly ash (w/c = 0.35)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	--	--	-0.002	0.111	-0.004	0.077
60	0.000	0.252	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.18 - Sulfate Expansions and Mass Changes for Batch 2FA-2a

Batch Name	2FA-2a					
Description	Type I-II (B) Cement + 20% Class F fly ash (w/c = 0.45)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.000	0.146	-0.003	0.130	-0.001	0.102
60	0.006	0.230	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.19 - Sulfate Expansions and Mass Changes for Batch 2FA-3a

Batch Name	2FA-3a					
Description	Type I-II (B) Cement + 20% Class F fly ash (w/c = 0.55)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	-0.004	0.176	0.000	0.124	0.005	0.077
60	--	--	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.20 - Sulfate Expansions and Mass Changes for Batch 2FA-1b

Batch Name	2FA-1b					
Description	Type I-II (B) Cement + 30% Class F fly ash (w/c = 0.35)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	-0.001	0.154	0.003	0.023	0.002	0.002
60	0.000	0.195	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.21 - Sulfate Expansions and Mass Changes for Batch 2FA-2b

Batch Name	2FA-2b					
Description	Type I-II (B) Cement + 30% Class F fly ash (w/c = 0.45)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.000	0.164	-0.001	0.038	0.001	-0.016
60	--	--	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

Table F.22 - Sulfate Expansions and Mass Changes for Batch 2FA-3b

Batch Name	2FA-3b					
Description	Type I-II (B) Cement + 30% Class F fly ash (w/c = 0.55)					
Curing Procedure	73°F until 3500 psi		Room Temperature (28D @ 73°F)		Accelerated (7D @ 73°F + 21D @ 100°F)	
Soaking Time (Days)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)	Avg. Expansion (%)	Avg. Mass Change (%)
30	0.001	0.071	-0.005	0.040	-0.003	0.002
60	--	--	--	--	--	--
90	--	--	--	--	--	--
120	--	--	--	--	--	--
150	--	--	--	--	--	--
180	--	--	--	--	--	--
270	--	--	--	--	--	--
360	--	--	--	--	--	--
450	--	--	--	--	--	--
540	--	--	--	--	--	--

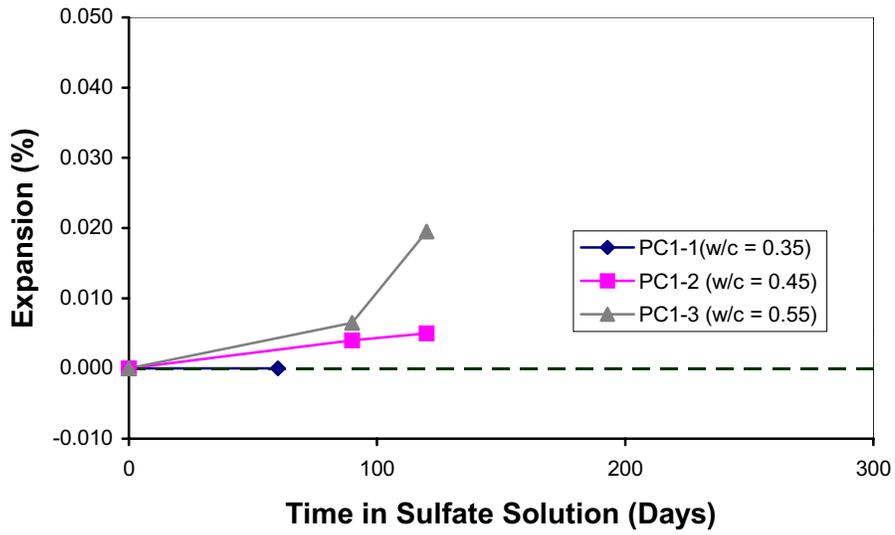


Figure F.1 - Sulfate Expansions of Plain Portland Cement Concretes with Type I Cement (Cured at Room Temperature to 3500 psi)

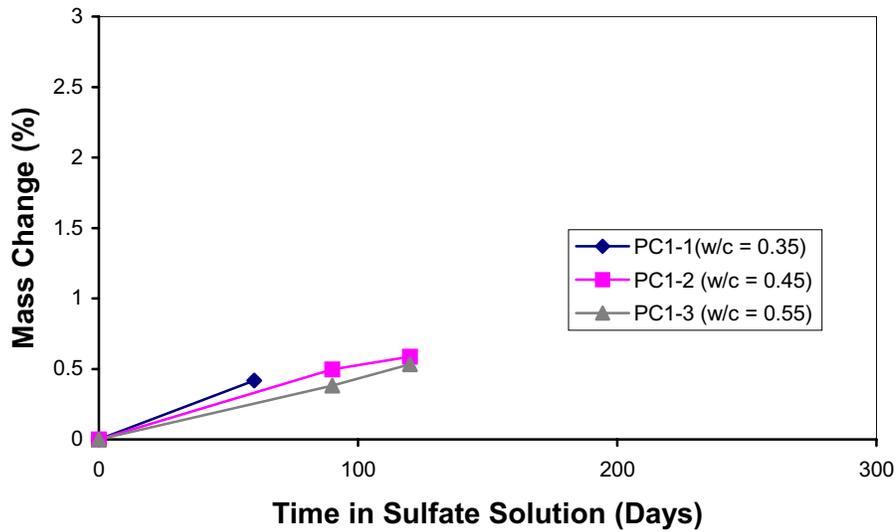


Figure F.2 - Mass Change of Plain Portland Cement Concretes with Type I Cement (Cured at Room Temperature to 3500 psi)

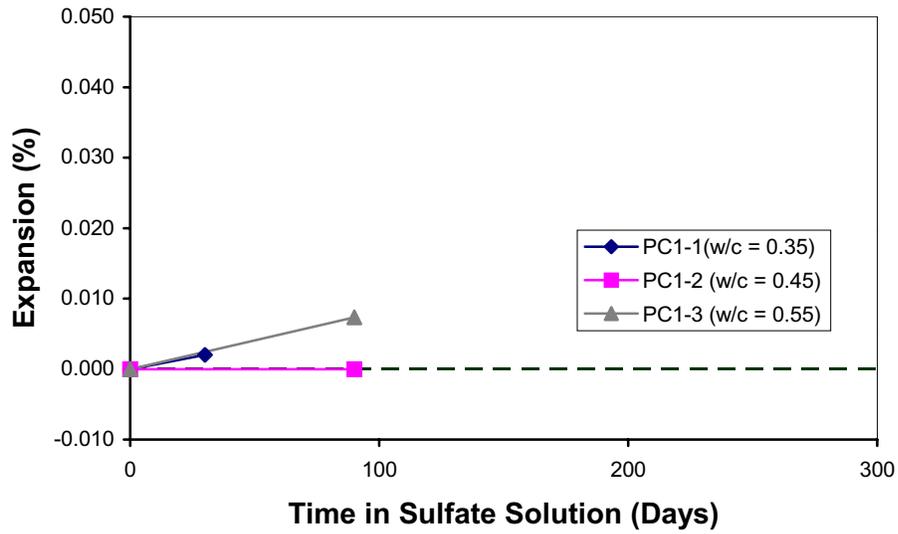


Figure F.3 - Sulfate Expansions of Plain Portland Cement Concretes with Type I Cement (Cured at Room Temperature for 28 Days)

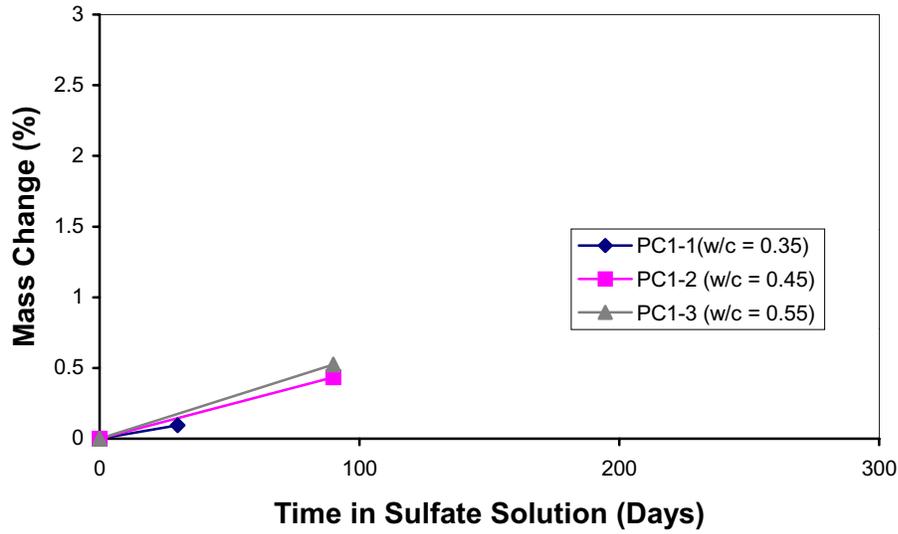


Figure F.4 - Mass Change of Plain Portland Cement Concretes with Type I Cement (Cured at Room Temperature for 28 Days)

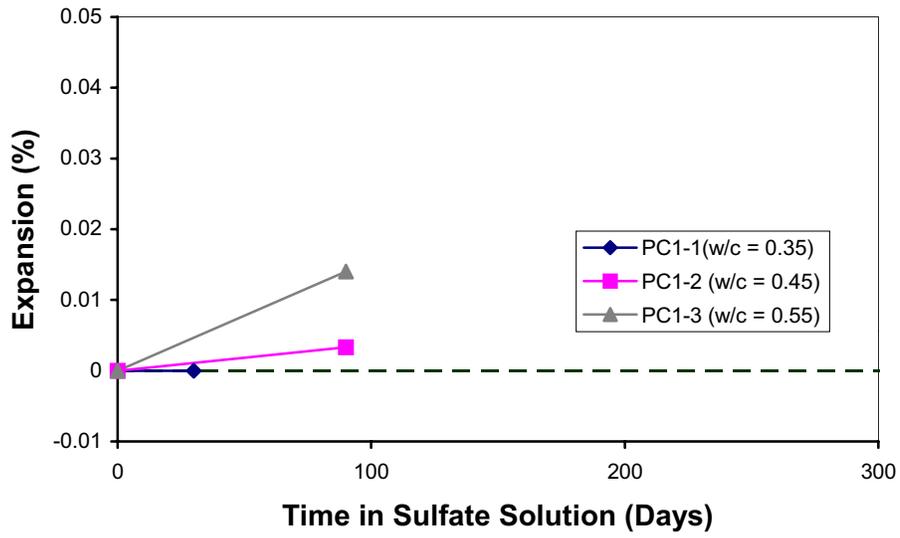


Figure F.5 - Sulfate Expansions of Plain Portland Cement Concretes with Type I Cement (Accelerated Cured for 28 Days)

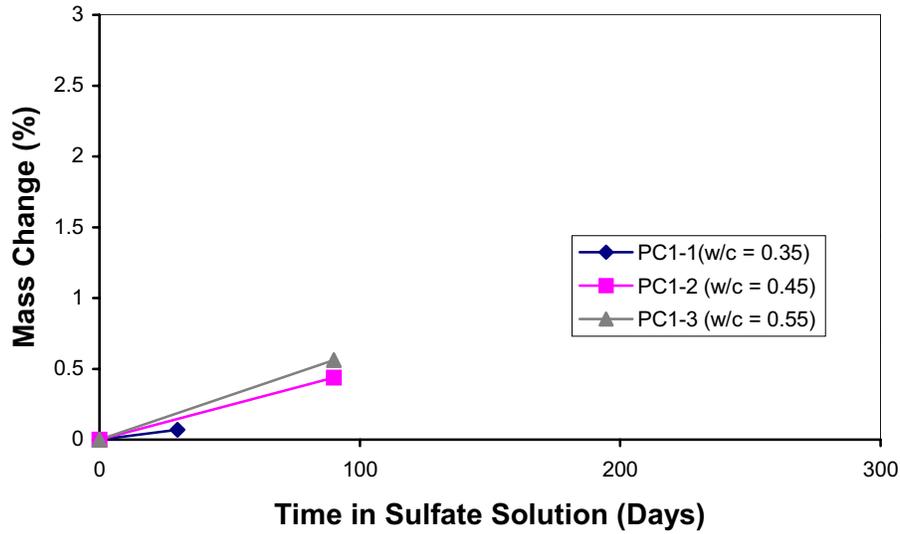


Figure F.6 - Mass Change of Plain Portland Cement Concretes with Type I Cement (Accelerated Cured for 28 Days)

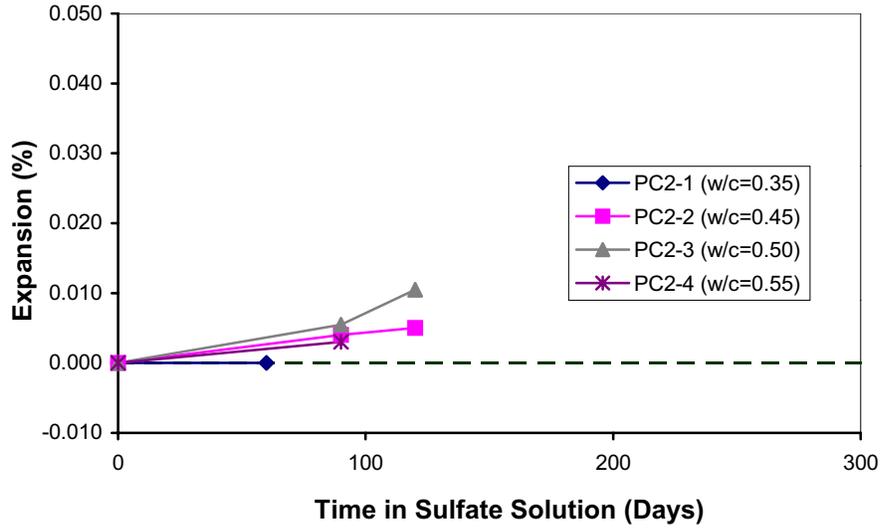


Figure F.7 - Sulfate Expansions of Plain Portland Cement Concretes with Type I-II (B) Cement (Cured at Room Temperature to 3500 psi)

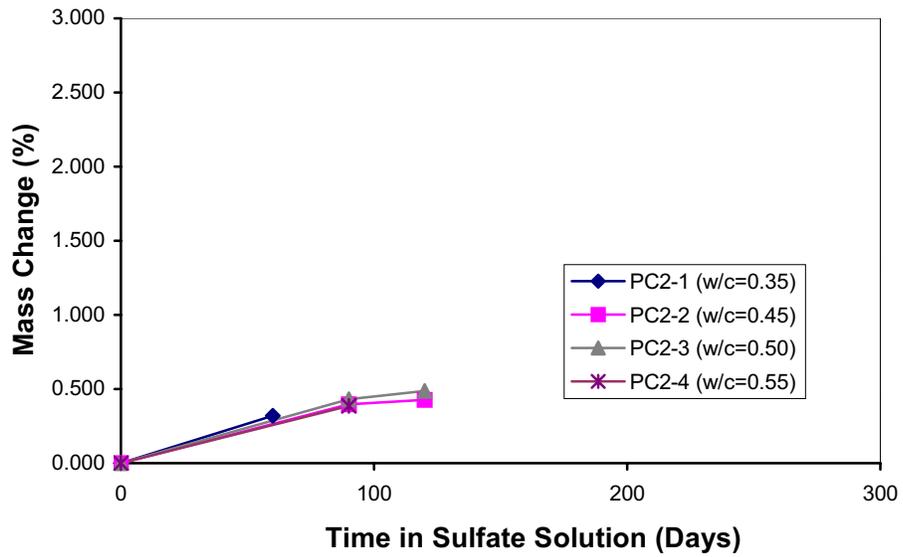


Figure F.8 - Mass Change of Plain Portland Cement Concretes with Type I-II (B) Cement (Cured at Room Temperature to 3500 psi)

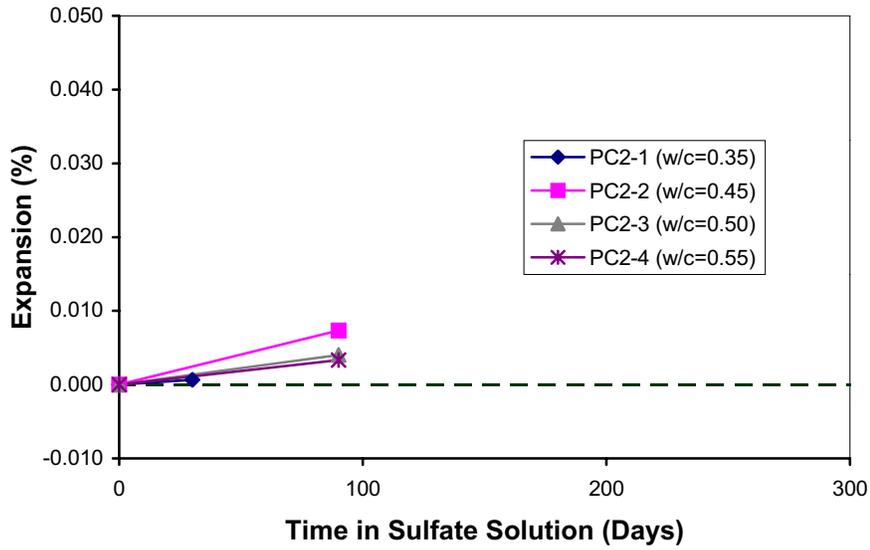


Figure F.9 - Sulfate Expansions of Plain Portland Cement Concretes with Type I-II (B) Cement (Cured at Room Temperature for 28 Days)

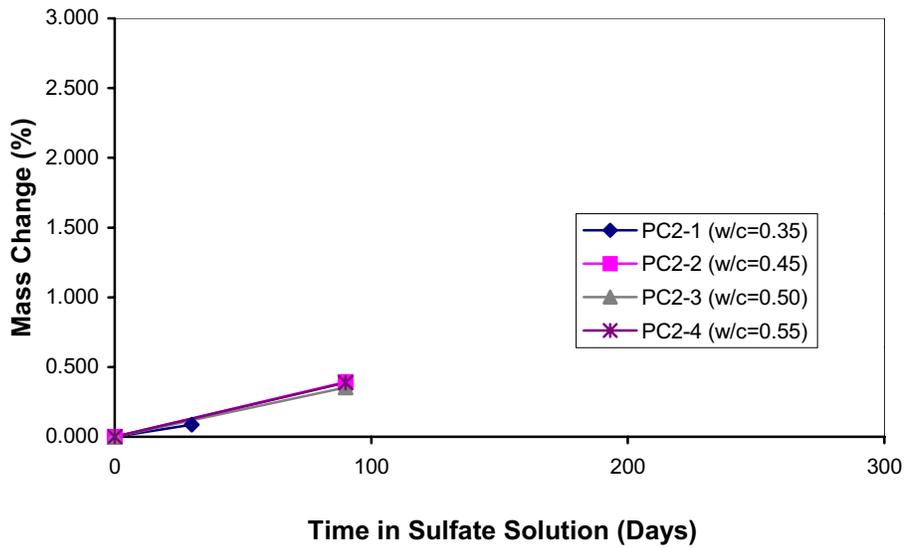


Figure F.10 - Mass Change of Plain Portland Cement Concretes with Type I-II (B) Cement (Cured at Room Temperature for 28 Days)

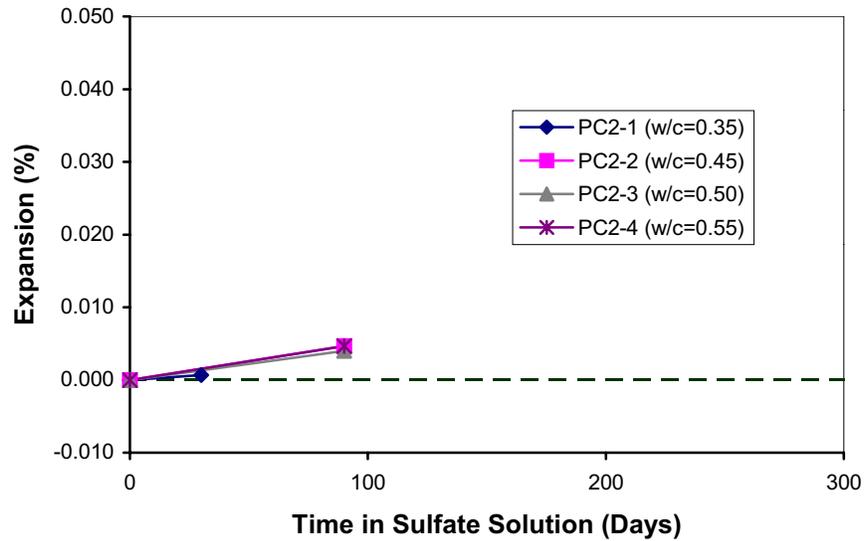


Figure F.11 - Sulfate Expansions of Plain Portland Cement Concretes with Type I-II (B) Cement (Accelerated Cured for 28 Days)

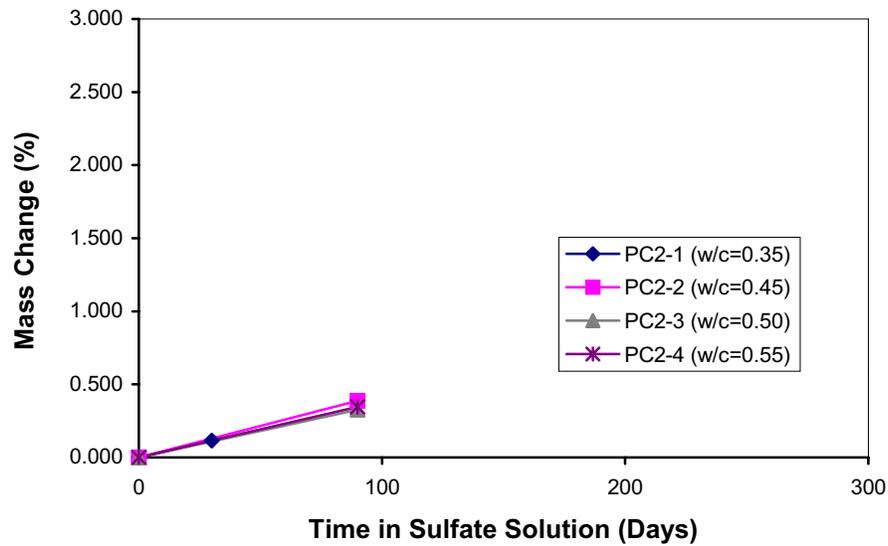


Figure F.12 - Mass Change of Plain Portland Cement Concretes with Type I-II (B) Cement (Accelerated Cured for 28 Days)

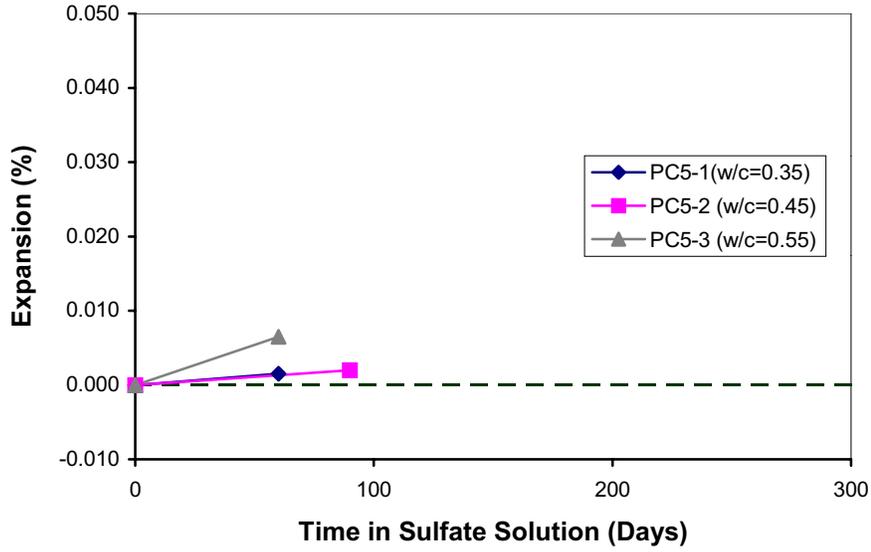


Figure F.13 - Sulfate Expansions of Plain Portland Cement Concretes with Type V Cement (Cured at Room Temperature to 3500 psi)

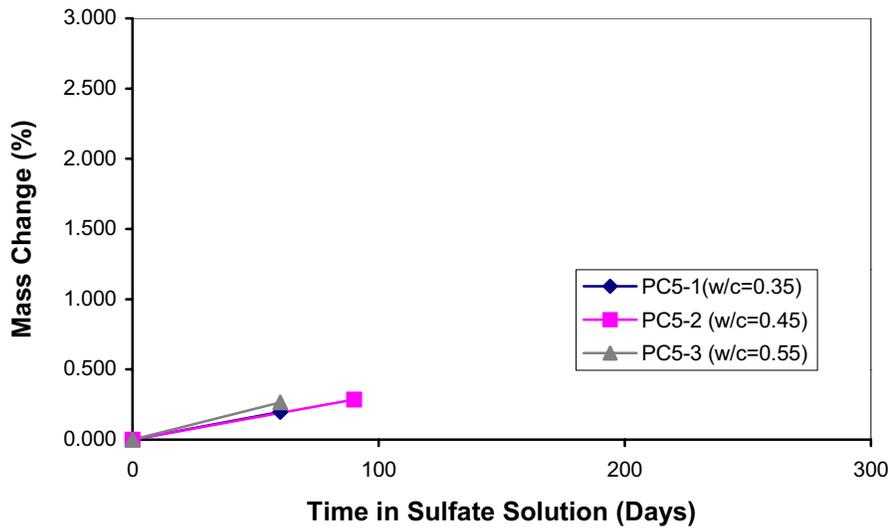


Figure F.14 - Mass Change of Plain Portland Cement Concretes with Type V Cement (Cured at Room Temperature to 3500 psi)

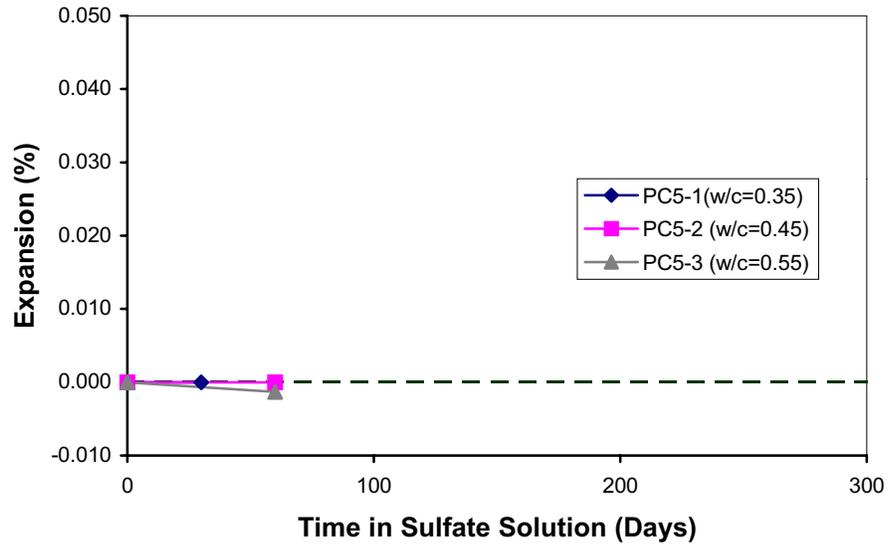


Figure F.15 - Sulfate Expansions of Plain Portland Cement Concretes with Type V Cement (Cured at Room Temperature for 28 Days)

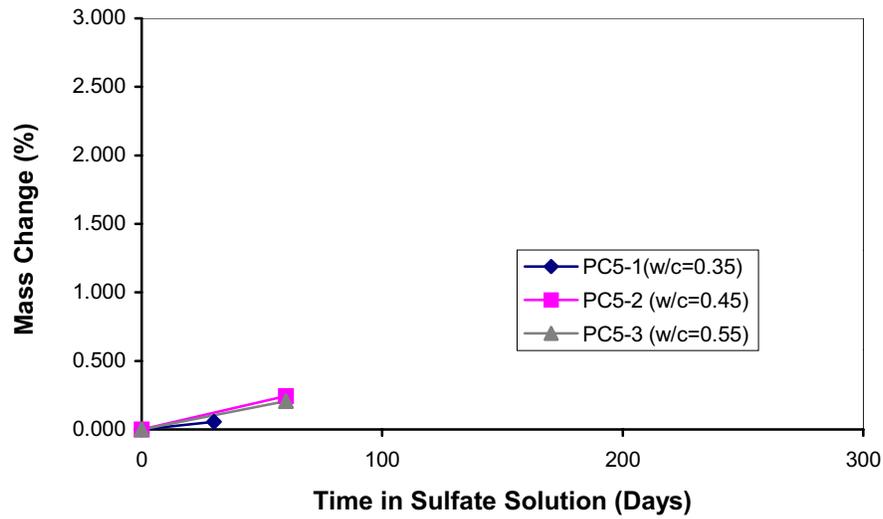


Figure F.16 - Mass Change of Plain Portland Cement Concretes with Type V Cement (Cured at Room Temperature for 28 Days)

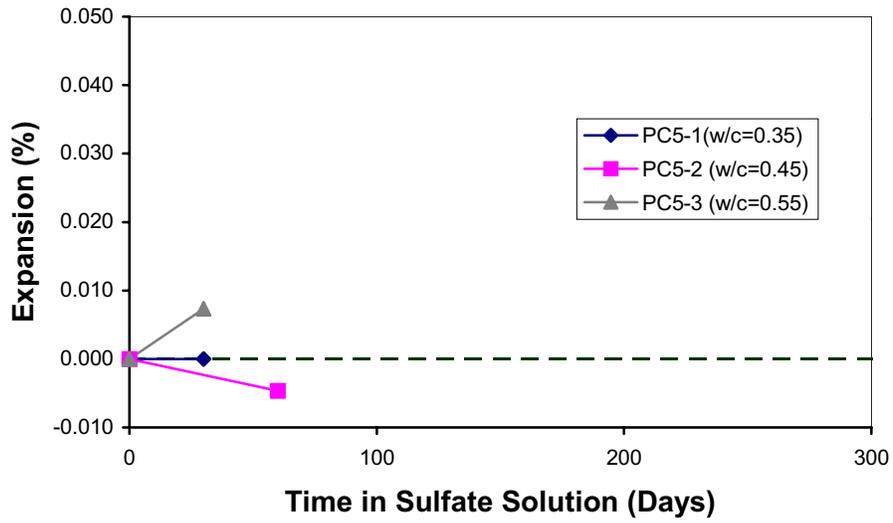


Figure F.17 - Sulfate Expansions of Plain Portland Cement Concretes with Type V Cement (Accelerated Cured for 28 Days)

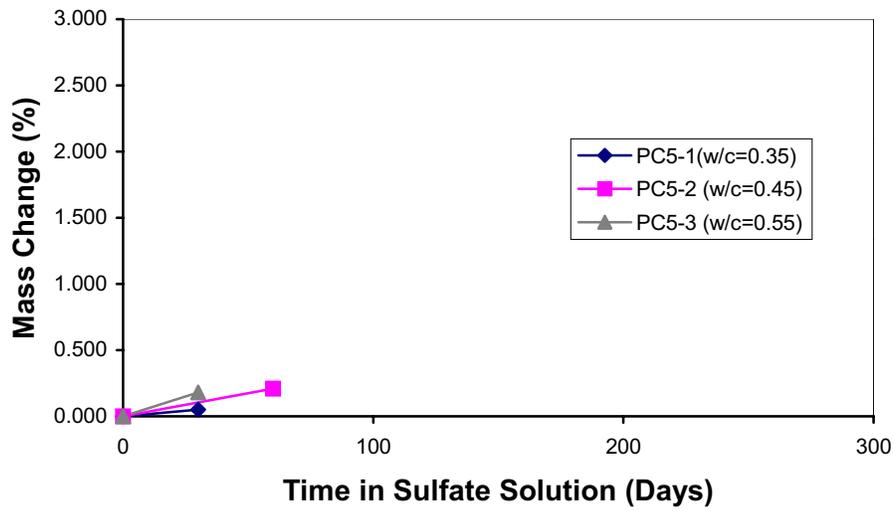


Figure F.18 - Mass Change of Plain Portland Cement Concretes with Type V Cement (Accelerated Cured for 28 Days)

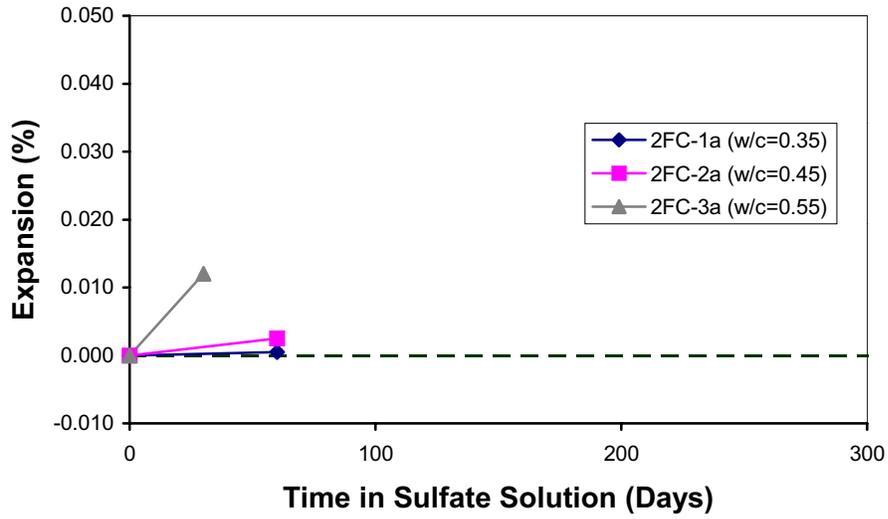


Figure F.19 - Sulfate Expansions of Concretes with 25 Percent Class C Fly Ash (Cured at Room Temperature to 3500 psi)

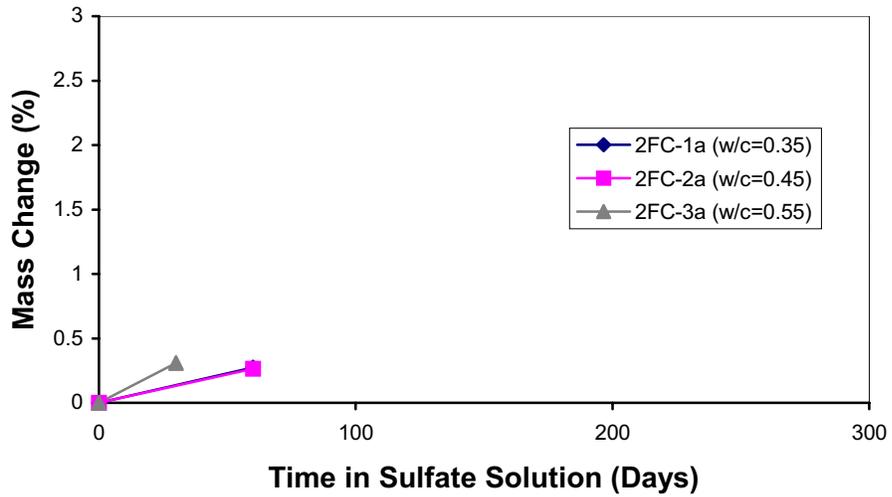


Figure F.20 - Mass Change of Concretes with 25 Percent Class C Fly Ash (Cured at Room Temperature to 3500 psi)

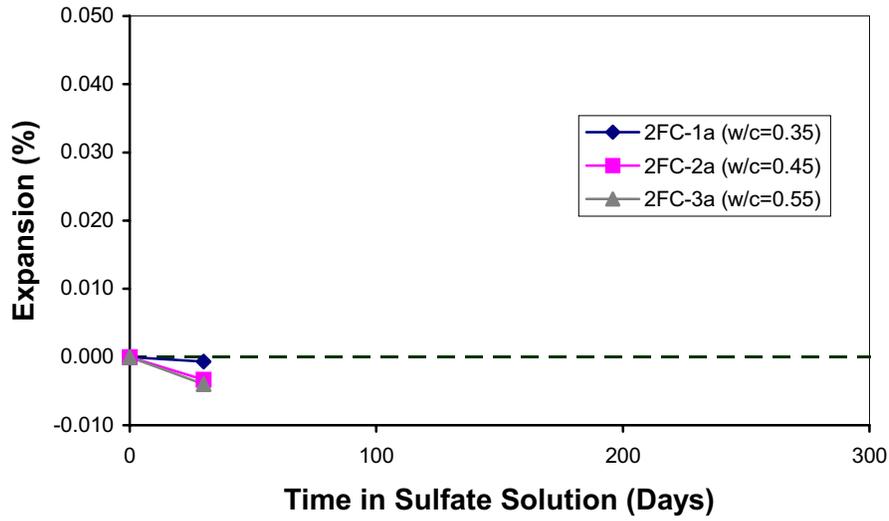


Figure F.21 - Sulfate Expansions of Concretes with 25 Percent Class C Fly Ash (Cured at Room Temperature for 28 Days)

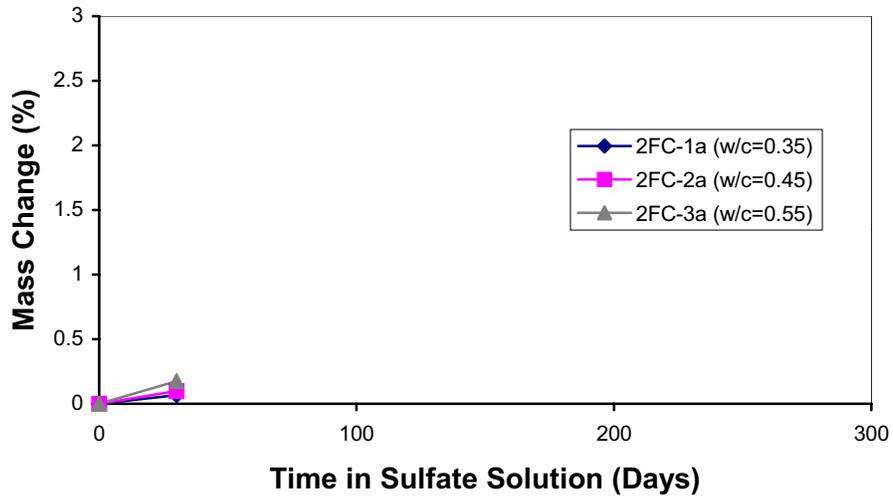


Figure F.22 - Mass Change of Concretes with 25 Percent Class C Fly Ash (Cured at Room Temperature for 28 Days)

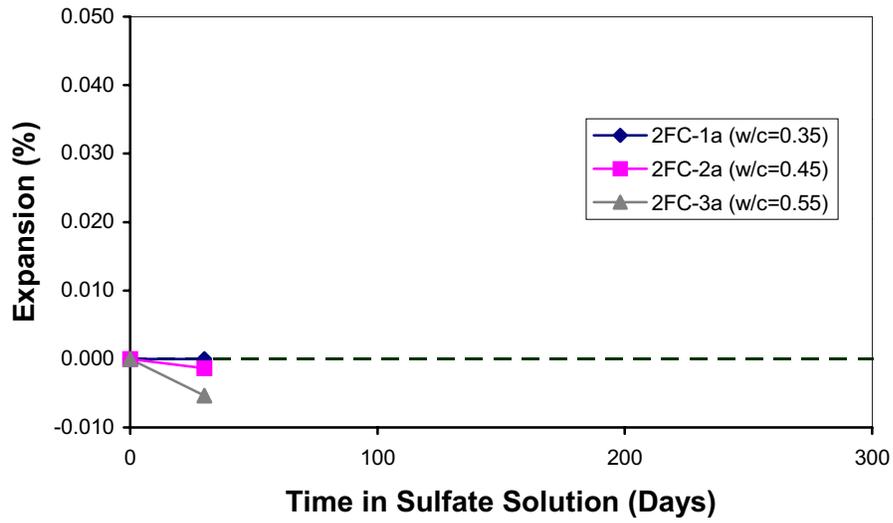


Figure F.23 - Sulfate Expansions of Concretes with 25 Percent Class C Fly Ash (Accelerated Cured for 28 Days)

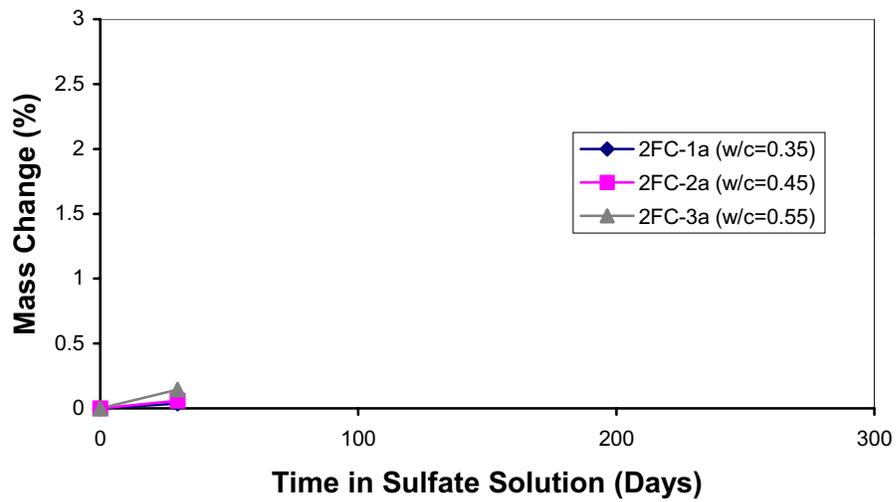


Figure F.24 - Mass Change of Concretes with 25 Percent Class C Fly Ash (Accelerated Cured for 28 Days)

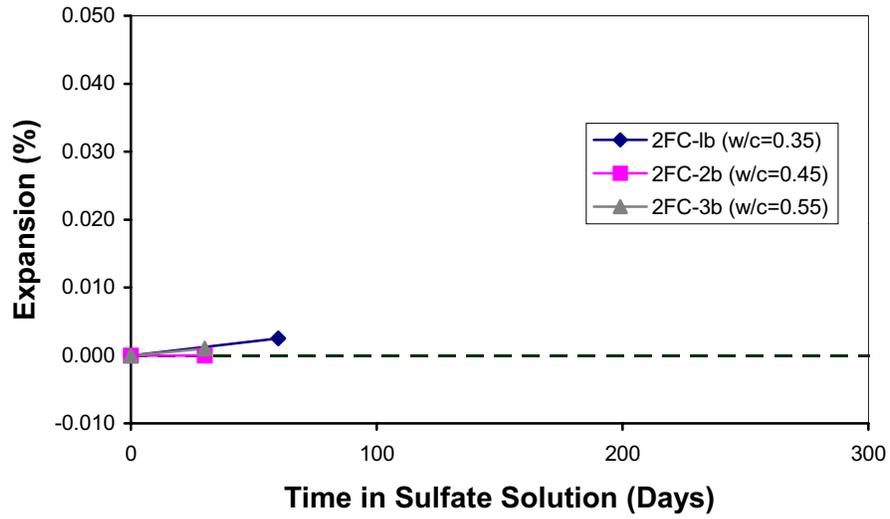


Figure F.25 - Sulfate Expansions of Concretes with 35 Percent Class C Fly Ash (Cured at Room Temperature to 3500 psi)

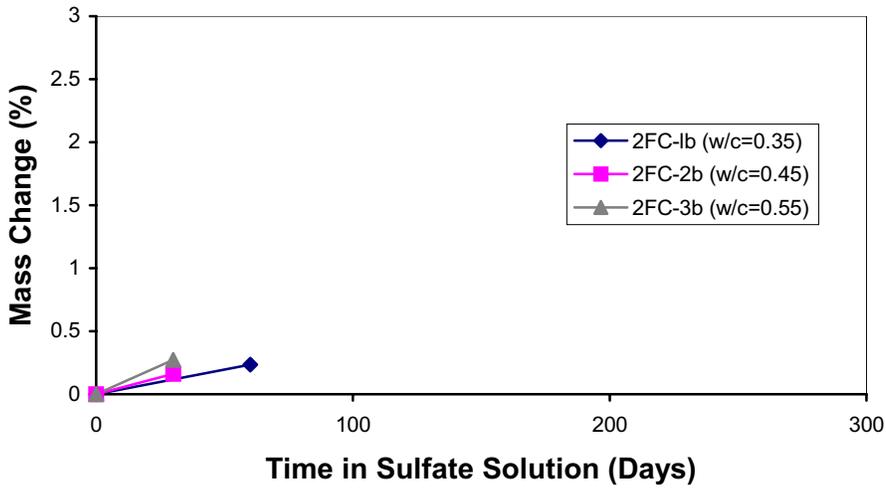


Figure F.26 - Mass Change of Concretes with 35 Percent Class C Fly Ash (Cured at Room Temperature to 3500 psi)

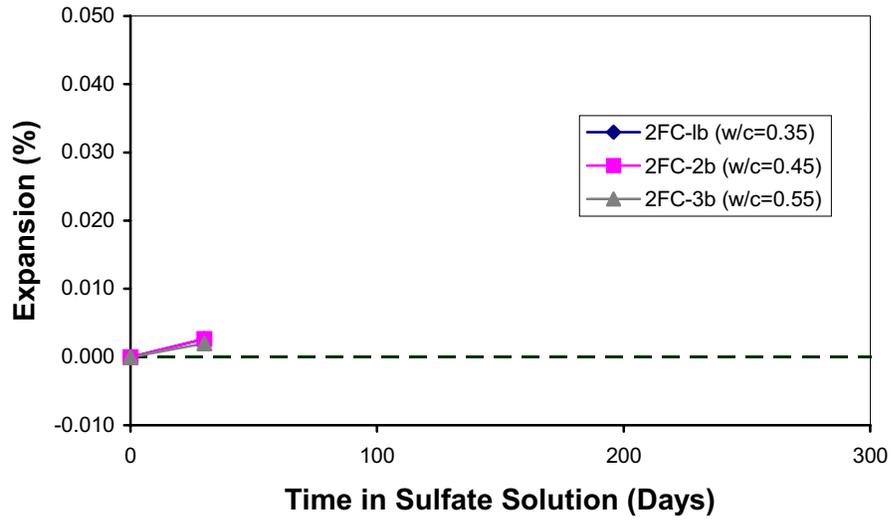


Figure F.27 - Sulfate Expansions of Concretes with 35 Percent Class C Fly Ash (Cured at Room Temperature for 28 Days)

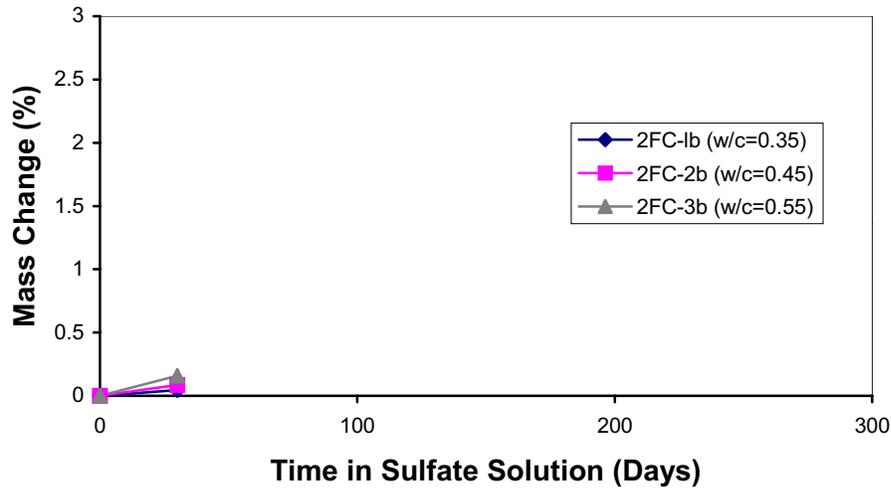


Figure F.28 - Mass Change of Concretes with 35 Percent Class C Fly Ash (Cured at Room Temperature for 28 Days)

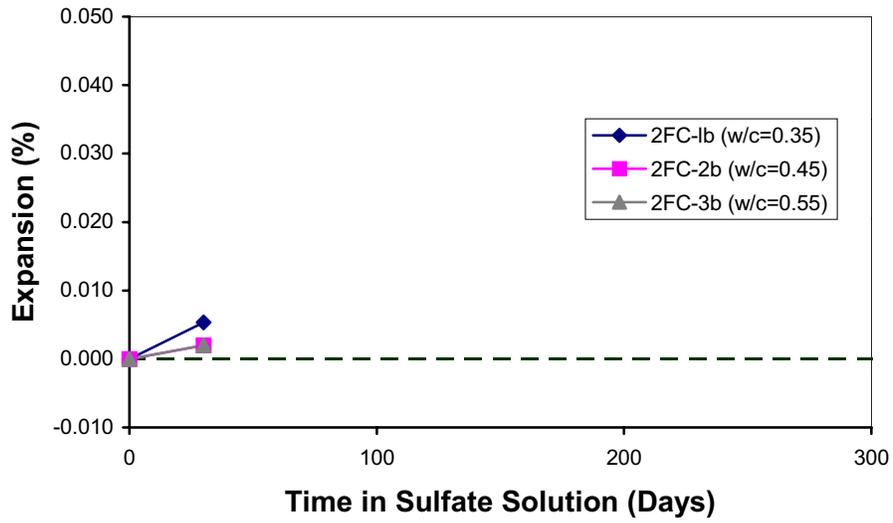


Figure F.29 - Sulfate Expansions of Concretes with 35 Percent Class C Fly Ash (Accelerated Cured for 28 Days)

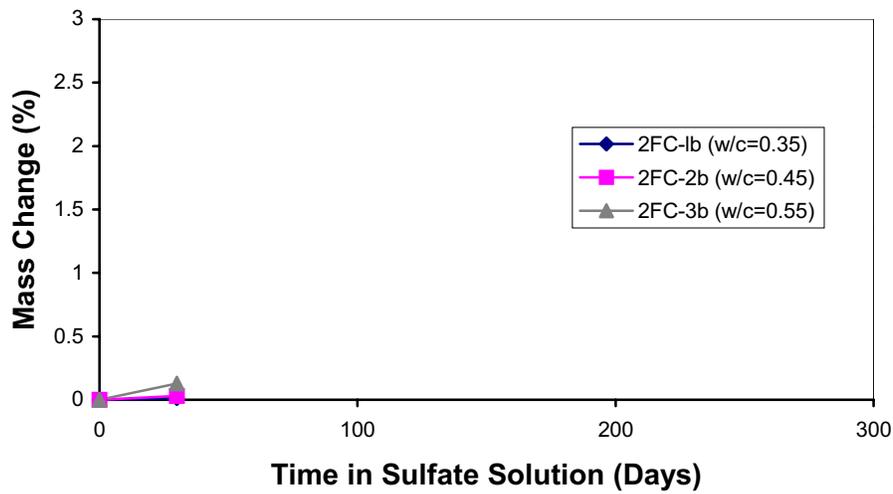


Figure F.30 - Mass Change of Concretes with 35 Percent Class C Fly Ash (Accelerated Cured for 28 Days)

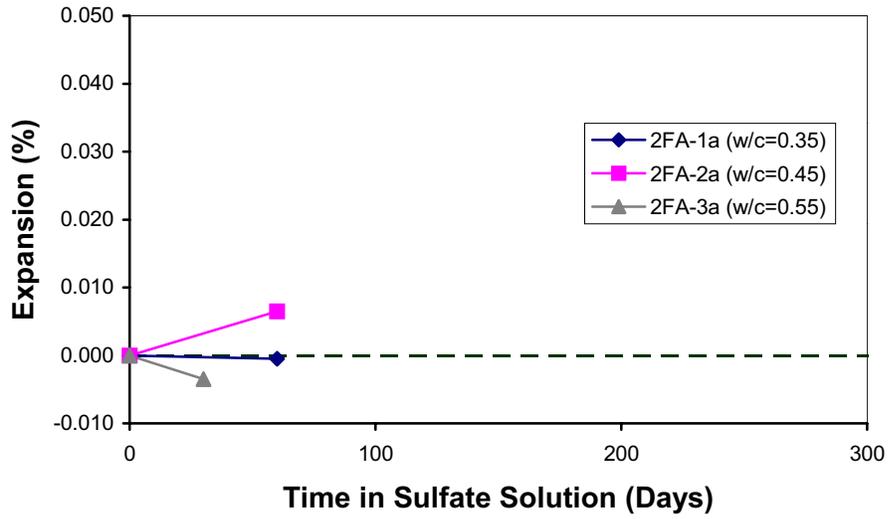


Figure F.31 - Sulfate Expansions of Concretes with 20 Percent Class F Fly Ash (Cured at Room Temperature to 3500 psi)

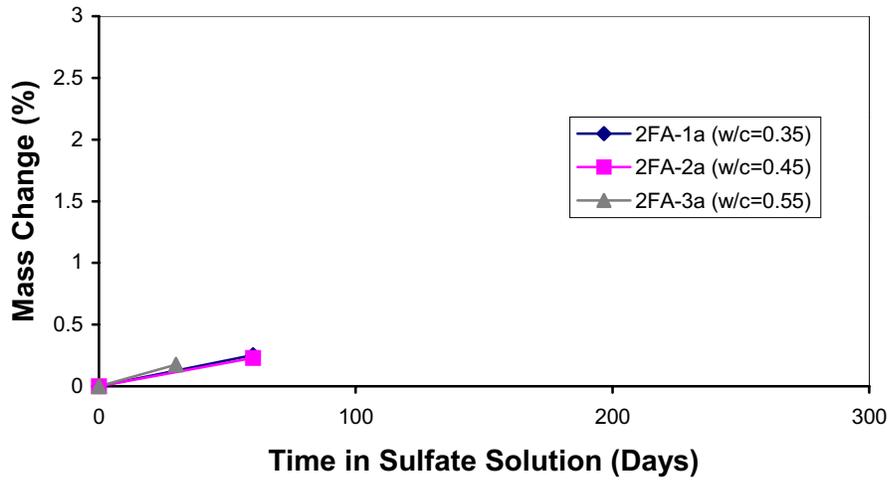


Figure F.32 - Mass Change of Concretes with 20 Percent Class F Fly Ash (Cured at Room Temperature to 3500 psi)

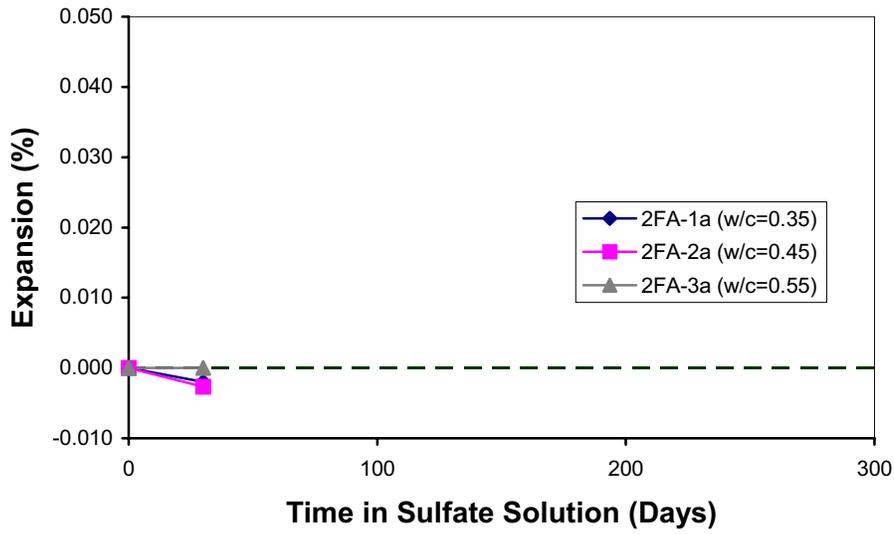


Figure F.33 - Sulfate Expansions of Concretes with 20 Percent Class F Fly Ash (Cured at Room Temperature for 28 Days)

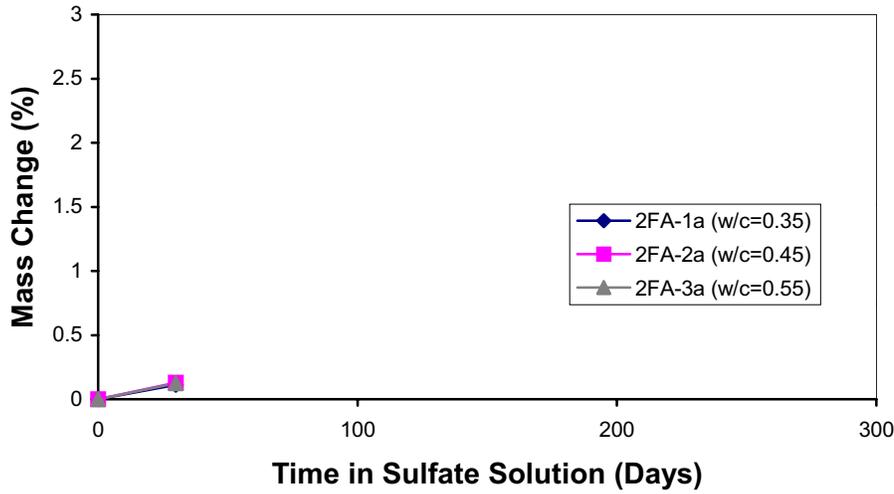


Figure F.34 - Mass Change of Concretes with 20 Percent Class F Fly Ash (Cured at Room Temperature for 28 Days)

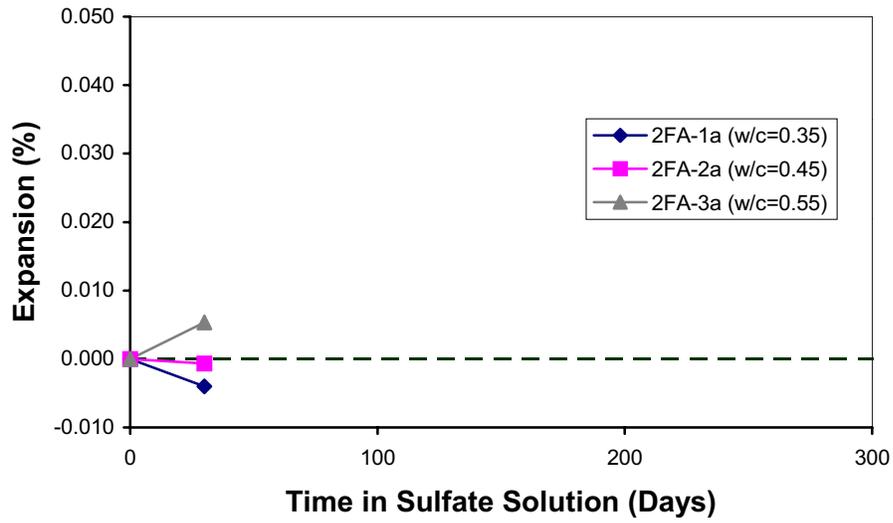


Figure F.35 - Sulfate Expansions of Concretes with 20 Percent Class F Fly Ash (Accelerated Cured for 28 Days)

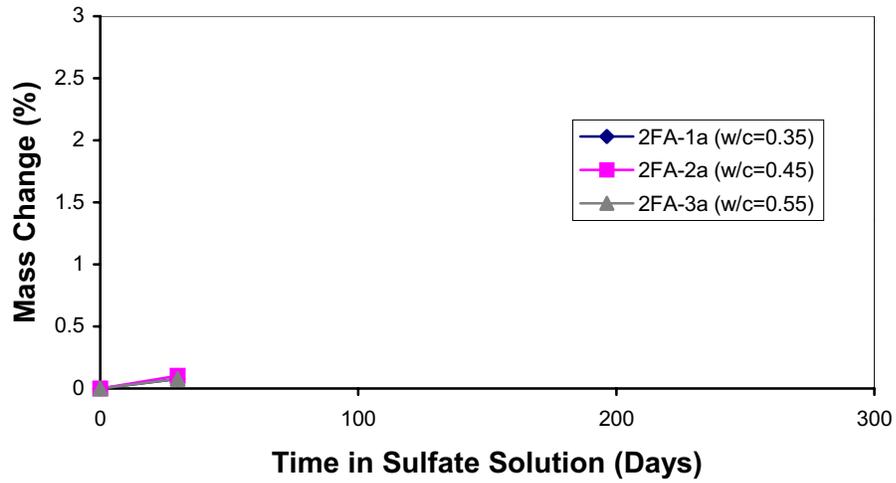


Figure F.36 - Mass Change of Concretes with 20 Percent Class F Fly Ash (Accelerated Cured for 28 Days)

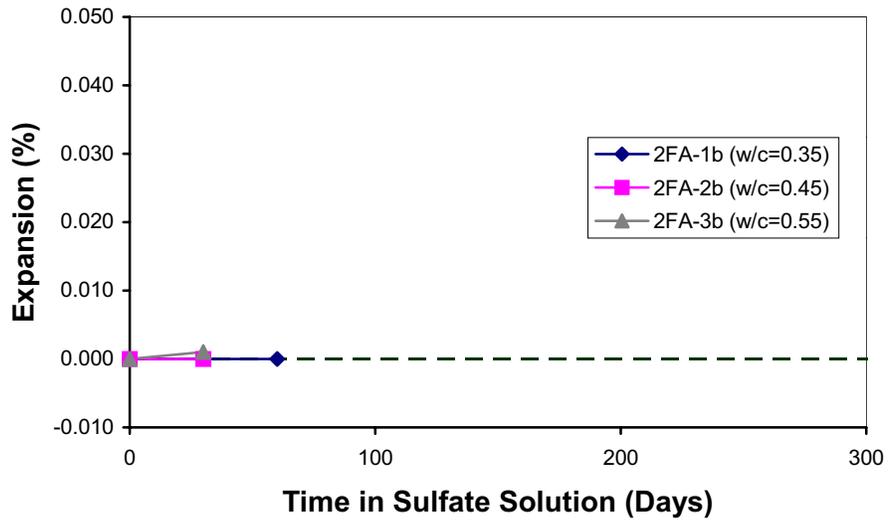


Figure F.37 - Sulfate Expansions of Concretes with 30 Percent Class F Fly Ash (Cured at Room Temperature to 3500 psi)

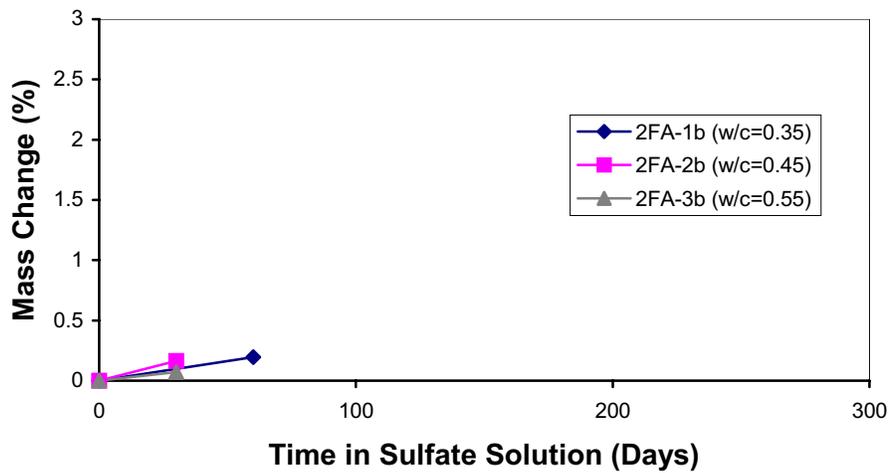


Figure F.38 - Mass Change of Concretes with 30 Percent Class F Fly Ash (Cured at Room Temperature to 3500 psi)

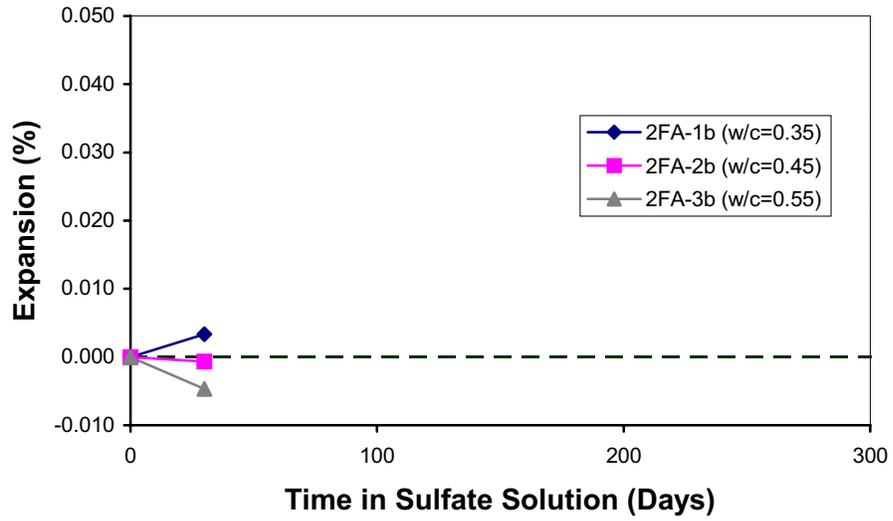


Figure F.39 - Sulfate Expansions of Concretes with 30 Percent Class F Fly Ash (Cured at Room Temperature for 28 Days)

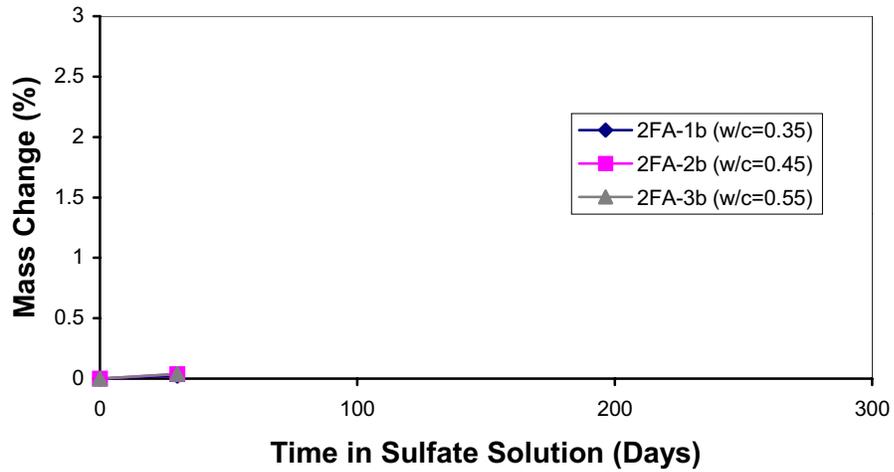


Figure F.40 - Mass Change of Concretes with 30 Percent Class F Fly Ash (Cured at Room Temperature for 28 Days)

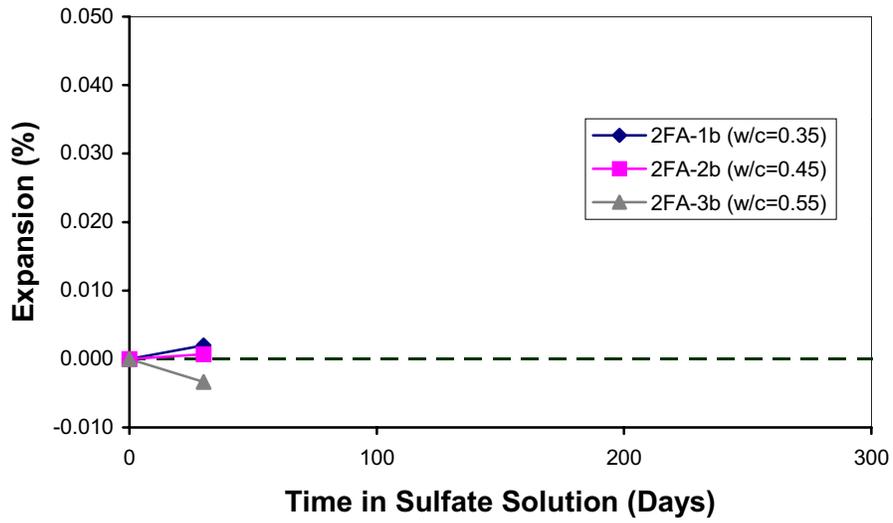


Figure F.41 - Sulfate Expansions of Concretes with 30 Percent Class F Fly Ash (Accelerated Cured for 28 Days)

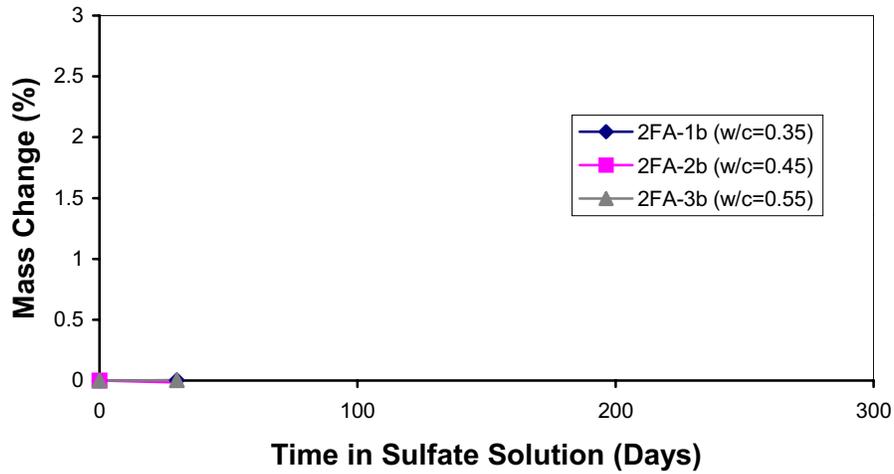


Figure F.42 - Mass Change of Concretes with 30 Percent Class F Fly Ash (Accelerated Cured for 28 Days)