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EVALUATION OF CDOT SPECIFICATIONS FOR CLASS H AND HT CRACK RESISTANT CONCRETE

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16. Abstract
This study examined the performance of concrete mixtures designed to increase cracking resistance for Colorado bridge decks. The current CDOT Class H and HT concrete mixtures and nine other mixtures were investigated to aid in the development of a more crack resistant concrete specification. A total of eleven concrete mixtures were designed, batched, and tested for their fresh and hardened concrete performance. Specifically, the designs differed by type of cement, w/cm, cement content, supplementary cementitious materials (SCMs), use of chemical admixtures, and aggregate type. Compressive strength, permeability, freeze-thaw resistance, and restrained shrinkage cracking were evaluated and documented in this report. Lower w/cm resulted in high early compressive strengths and rates of strength and strain development. Increasing the w/cm to 0.44 and Class F fly ash replacement levels up to 30% was beneficial in controlling strength gain. A low cement content mixture with increased w/cm and fly ash replacement proved to be beneficial. When SCMs were not utilized, a low cement content of 6.0 bags was beneficial. When SCMs were used, increased cement content helped to maintain the same properties. Type G, coarse-ground cement was beneficial to strain and strength at the higher w/cm of 0.42 and low cementitious materials content. At a lower w/cm of 0.38, the mixture behaved similarly to the control mixture fabricated using Type II cement, developing strain and strength at an average rate.

A high dosage rate of a shrinkage reducing admixture was extremely beneficial in controlling both the development rate and ultimate strain of the mixture, while maintaining adequate development of ultimate strength at all ages. An average dosage rate of a set retarder only retarded the initial strength development slightly. After 1 day of age, the development of strength and strain was substantially increased. Although the concrete containing the set retarder reached higher compressive strengths more quickly than anticipated, the concrete did not crack in the AASHTO PP34 test and was moderately durable.

Implementation:
To implement this research: increase maximum allowable w/cm from 0.42 to 0.44; increase maximum allowable cement replacement with Class F fly ash from 20-30%; allow the use of cement replacement with ground-granulated blast furnace slag up to 50%; incorporate the use of a shrinkage reducing admixture at high dosage rates; incorporate the use of a set retarder admixture at average dosage rates; and decrease cementitious content to 564 lb/cy when supplementary cementitious materials are not used.

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EXECUTIVE SUMMARY

Within the past five years, the Colorado Department of Transportation (CDOT) has experienced a continued problem with cracking of bridge decks. In 2003, CDOT implemented concrete mixture designs Class H and Class HT into the CDOT *Standard Specification for Road and Bridge Construction*. Class H and HT were developed to provide crack resistant concrete structures and were intended to be used in the construction of bridges and other concrete structures. Recently, the CDOT has noticed cracking in several bridge decks using these concrete specifications.

This research includes the design and testing of over ten concrete mixtures in an effort to create a more crack resistant concrete than the current CDOT Class H and HT concrete specification. Cracking is known to be the result of many factors including shrinkage. The concrete mixtures designed for this research were designed with water-to- cementitious (w/cm) material's amounts and cement replacement percentages both above and below the current specifications. The design approach was intended to investigate the effect of individual and multiple supplemental cementitious materials replacement levels on the fresh and hardened concrete properties: restrained shrinkage strain, compressive strength, rate of strength gain, freeze/thaw durability, and permeability.

A national state Department of Transportation (DOT) survey was conducted and offered to each state's bridge and/or materials engineers. They were queried regarding their state's current and past research involving crack resistant concrete as well as comments on their state DOT specifications currently used for bridge decks. The results of this survey were used in during the experimental portion of this study to aid in improving the current CDOT Class H and HT specifications.

A more crack resistant concrete mixture was developed through this study. The recommendations made within provide the CDOT with the necessary information to produce more durable and crack resistant concrete bridge decks. The primary recommendations from this research are: increase the maximum allowable w/cm, decrease the cementitious content, increase the percent of allowable fly ash, and include a shrinkage reducing admixture.

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CHAPTER 1 – INTRODUCTION

1.1 Concrete

1.1.1 Problematic Cracking in Concrete

Within the past five years, the Colorado Department of Transportation (CDOT) has experienced a continued problem with cracking of bridge decks. In 2003, CDOT implemented concrete mixture designs Class H and Class HT into the CDOT *Standard Specification for Road and Bridge Construction*. Class H and HT were developed to produce crack resistant concrete structures and were intended to be used in the construction of bridges and other concrete structures (CDOT, 2005). Recently, the CDOT has noticed cracking in several bridge decks using these concrete specifications. CDOT and other state DOT's are interested in low-cracking potential concrete in an effort to reduce maintenance costs and delays to the motoring public. Ultimately, the primary objective is to improve the performance of concrete bridge decks in Colorado by minimizing cracking potential of the concrete mixtures used in them.

Cracking in reinforced concrete structures allows water and contaminants to migrate inside the structure where it can cause deterioration of the reinforcing steel as well as the surrounding concrete. Water that is able to penetrate through the bridge superstructure can also cause damage to the substructure and affect bridge aesthetics. The deicing chemicals used during inclement weather to provide safe driving conditions in combination with air and water accelerates the corrosion of reinforcing steel (rust or oxidation). The existing bond between the concrete and the steel diminishes as the corrosion process progresses, jeopardizing the integrity of the structure. When a bridge is in service and experiences cracking, naturally the cracks grow with time. This allows for more water and deicing chemicals to enter the deck and degrade the reinforcing steel, creating the need for replacement or repair earlier than normal. This perpetuation of bridge deterioration requires costly and labor-intensive repair.

To minimize the amount of cracking and reduce maintenance costs, Class H and HT concrete mixtures were analyzed in this study to ensure the concrete meets the expectations of the CDOT. Further, additional mixtures were evaluated for their effectiveness in reducing cracking potential in concrete structures. To accomplish this, eleven concrete mixtures were designed with low-cracking potential as the primary objective. The results of this study and recommendations are included in this report.

CHAPTER 2 - BACKGROUND

2.1 Colorado Department of Transportation

2.1.1 Research Interest

In 2003, the Colorado Department of Transportation (CDOT) revised their *Standard Specifications for Road and Bridge Construction* to include two new classes of structural concrete. Class H and Class HT concrete were included into the standard specifications as a crack resistant concrete. These concretes are currently used in the construction of bridges and other concrete structures. Class H concrete is used for concrete bridge decks without a topping slab and waterproofing membrane [Xi et. al, 2003]. Class HT concrete is used as a top layer for exposed concrete bridge decks. The design criterion for each of these concrete classes is shown in Table 2.1.

Table 2.1 Class H and Class HT Mixture Specifications

Material	Class H	Class HT
Cement [C]	450 - 500 lbs/yd^3	450 - 500 lbs/yd^3
Fly Ash [FA]	90 - 125 lbs/yd^3	90 - 125 lbs/yd^3
Silica Fume [SF]	20 - 30 lbs/yd^3	20 - 30 lbs/yd^3
C + FA + SF	580 - 640 lbs/yd^3	580 - 640 lbs/yd^3
Course Aggregate	AASHTO M 43 Size No. 67> 55%	AASHTO M 43 Size No. 7 of 8 > 55%

A study on Colorado bridge decks was published in March 2003 [Xi et al, 2003]. The objectives of this study were twofold. First, the extent and causes for bridge deck cracking was investigated. Secondly, concrete material properties, construction practices, and design specifications where examined as to possible causes for bridge deck cracking. A literature review within this study concluded that cracking in early age bridge decks is a result of material, design, construction, and environment. High early age shrinkage was found to be a major cause for this cracking problem. In addition, the structural design had a direct role in cracking as well. Cracks were typically noticed above girders and piers. Placement and curing can have a significant role in cracking, primarily plastic shrinkage cracking. Recommendations regarding materials, design factors, and construction practices were included in the final report. Cement and silica fume content, water/cement ratio, and the rate of strength gain were key recommendations regarding materials included in the report.

Recently, the CDOT has discovered a number of bridge decks throughout the state constructed with Class H and Class HT concrete that exhibit cracking. It is suspected that the rate of strength gain for these concrete mixtures may in part be a contributing factor to this cracking. Several bridge decks have obtained the 28-day compressive strength within three days. Other factors that influence cracking include: types and amount of aggregate, cement content and type, water/cement ratio, and air content. These are discussed in more detail in Chapter 3 of this report.

Colorado's harsh weather conditions make it essential for the states bridge decks to have strict performance and mixture specifications. Early age cracking of bridge decks can decrease the life of the structure and increase maintenance costs.

2.1.2 Current Specifications

2.1.2.1 Class H Specifications

Class H concrete is used for bare concrete bridge decks with no waterproofing membrane. Below is a summary of current CDOT Class H and HT specifications.

- 56-day compressive strength of 4500 lbs./in.^{2};
- Required air content of 5% - 8%;
- Water-to-Cementitious Ratio (w/cm) ranging from 0.38 – 0.42;
- An approved water reducing admixture ;
- A minimum of 55 percent AASHTO M 43 size No. 67 coarse aggregate by weight of total aggregate;
- Laboratory trial mixture must not exceed permeability of 2000 coulombs at 56-days of age (ASTM C 1202) and must not exhibit a crack at or before 14 days in the cracking tendency test (AASHTO PP 34).

2.1.2.2 Class HT Specifications

The CDOT Class H and HT concrete have identical specifications and are used for bare concrete bridge decks that will not receive a waterproofing membrane. The difference between the Class H and HT lies in that Class HT concrete is used as the top layer of the bare bridge deck. The specifications for the CDOT Class HT concrete are summarized below:

- 56-day compressive strength of 4500 lbs./in.^{2};
- Air content of 5% - 8% are required;
- W/cm ranging from 0.38 – 0.42;
- An approved water reducing admixture;
- Must have a minimum of 50 percent AASHTO M 43 size No. 7 or No. 8 coarse aggregate by weight of total aggregate
- Laboratory trial mixture must not exceed permeability of 2000 coulombs at 56-days (ASTM C 1202) and must not exhibit a crack at or before 14 days in the cracking tendency test (AASHTO PP 34).

2.2 Cracking in Concrete

2.2.1 Importance of Cracking in Concrete

Concrete is known to be weak in tension. In design, concrete beams are assumed to have zero tensile strength. These tensile stresses are fairly low when compared to those experienced by reinforced bridge decks or beams, which spans are restrained between two or more supporting structures. Individual lanes of bridge decks are sometimes placed while others on the bridge remain open for service. For many reasons, bridge decks experience movement (deflection) during daily traffic and thermal expansion which can contribute to the concrete cracking. The earlier the concrete deck cracks the faster the rate of deterioration and need for repair. As a result, the concrete must be more durable and designed to have characteristics that will be advantageous during early ages and in this environment. A decrease in early age cracking will delay the development of corrosion on the steel reinforcement, decreasing its permeability and increasing the structures durability.

Cracking in reinforced concrete structures allows water and contaminants to migrate inside the structure where it can cause deterioration of the reinforcing steel as well as the surrounding concrete. In addition, water that is able to penetrate through the bridge superstructure can cause damage to the substructure and affect bridge aesthetics.

To minimize the amount of cracking and reduce maintenance costs, Class H and HT concrete mixtures were analyzed in this study to ensure the concrete meets the expectations of the CDOT. Further, additional mixtures were evaluated for their effectiveness to eliminate or at least reduce cracking in concrete structures.

2.3 Causes of Cracking in Concrete

2.3.1 Internal Stresses

Concrete cracks as the result of numerous factors. Internal stresses within the concrete are the primary cause of early-age cracking. Internal stresses develop depending upon the heat of hydration, the rate of strength gain, 28-day and 56-day compressive strength, cement content, percent replacement of cement with supplementary cementitious materials (SCMs), and w/cm (Equation 1);

$$w/cm = \frac{water}{cementitious\ materials} \qquad \textbf{Eq. 1}$$

Additionally, the use of chemical admixtures is necessary to create various desirable characteristics of the mixture. These characteristics include reducing shrinkage, delayed set time and air content. All of which can impact the magnitude and rate of development of internal stresses and cause cracking.

2.3.2 External Stresses and Normal Use Degradation

Daily, cyclic service loading is a major cause of cracking in concrete bridge decks. These stresses are unavoidable as the Colorado weather, temperature fluctuation, and traveling vehicles gradually degrade the roadways and deck surfaces.

2.3.3 Restraint

Restraint has long been an issue regarding bridge deck cracking. Deck slabs are restrained against movement at joints and internally around steel reinforcement. As concrete expands thermally or shrinkage occurs, the restraint against movement will result in cracking. Expansion joints in bridges help to alleviate cracking due to these stresses.

2.3.4 Shrinkage Strain

Shrinkage strain is a major cause of early age cracking in concrete and the primary focus for this research. Multiple types of shrinkage exist and are all detrimental to the life of the concrete. As water leaves the cement paste matrix, the cement paste begins to reduce in volume and is termed ‘shrinkage.’

Drying shrinkage represents the strain caused by the loss of water from hardened concrete. This type of shrinkage results in surface cracking (map-cracking) and causes the surface of the bridge deck to deteriorate at a much faster rate.

A type of drying shrinkage is termed autogenous shrinkage, which occurs as the internal water is gradually depleted during the continued hydration of cement particles over the life of the concrete. Regardless of the type of shrinkage, the volume of the cement paste has a tendency to shrink as the water dissipates. Shrinkage begins to occur immediately after the concrete sets, as surface water begins to evaporate and with the continued hydration of cement particles. The voids in the concrete once occupied by water are then left empty. The volume shrinkage that attempts to occur within the rigid cement paste matrix creates internal stresses within the concrete. These stresses induce a strain on the concrete that results in early age cracking. This research utilizes the AASHTO P34 Restrained Ring Shrinkage Test to measure these shrinkage strains versus time. The primary objective of this research is to design, batch, and test a minimum of ten concrete mixtures to examine various aspects of concrete mixtures and their influence on cracking. Specifically, this research aims to develop a concrete mixture that is more resistant to cracking than the current Class H and HT specification. A more detailed explanation and understanding of the tests performed for this research is included in the literature review in Chapter 3 of this report.

2.4 Research Objectives

2.4.1 Objectives of Investigation

The primary objectives of this study are to design a more crack resistant concrete for use in Colorado's bridge decks. The benefit that is gained from this research is that the CDOT is in a better position to design and construct crack resistant bridge decks and other concrete structures. Results from this study provide the necessary information to develop more durable concrete bridge decks. The recommendations within will allow the CDOT to make changes to the current specification for future construction.

Ancillary benefits from this study will include a cost savings to the CDOT. With the consideration of the recommendations of this study, a more crack-resistant concrete will benefit the CDOT by providing for longer lasting concrete structures and reduce annual costs to maintain these pavement structures.

CHAPTER 3 - LITERATURE REVIEW

3.1 Preface

This literature review does not examine the effects of superstructure design on concrete bridge deck cracking. Construction practices such as curing, finishing, time of placement (ambient temperature), and consolidation play a major role in bridge deck cracking. This study investigates the effect of mixture design factors which influence bridge deck cracking. Curing practices are discussed herein only to emphasize its importance in the practice of placing and producing durable concrete.

3.2 Curing

Curing is not the focus of this study; however, curing is essential to producing quality concrete. Curing is the method used to reduce the evaporation of water immediately after placement and is required to promote continued hydration of the cement, thereby increasing the concrete's compressive strength and overall durability. The effect of curing cannot be neglected in practice. Furthermore, the effect of curing on compressive strength and shrinkage cannot be disregarded. All of the research examined for this literature review discusses the importance of adequate curing. Internal curing is the only method of curing pertaining to the scope of this research and is discussed in further detail in Section 3.4.7.3.3

3.3 Concrete Shrinkage

Shrinkage is a major cause of cracking in concrete bridge decks. When cement is hydrated and water evaporates, internal stresses develop and volume shrinkage of the concrete occurs, autogenous shrinkage and drying shrinkage, respectively. The hardened concrete attempts to resist these stresses and cracks as a result. A concrete mixture design may combat shrinkage by adjusting the quantity of any one or multiple materials used in making concrete. A literature review was conducted on several available studies involving cracking in concrete bridge decks. The research information reviewed was built upon in an effort to efficiently provide the CDOT with revised and more durable bridge deck mixture designs.

3.3.1 Effect of Restraint on Shrinkage

Restraint has long been known to cause bridge deck cracking. As a concrete bridge deck dries and moisture evaporates, it experiences a volume decrease termed shrinkage. According to Krauss and Rogalla (1996), the amount of shrinkage depends primarily on the paste content and water content. Reinforcement and the bridge superstructure components such as girders provide restraint against shrinkage, resulting in tensile stresses that cause the concrete to crack (1996). Restrained ring shrinkage tests (AASHTO PP34, ASTM C 1581) allow researchers to conduct a relative comparison of the micro strain associated with different mixture materials at the point of cracking due to restraint in a controlled environment. Cracking is indicated as the point when the strain in the steel ring suddenly decreases. The exposed surface of the concrete ring makes inspection for cracks simple although several mixtures did not exhibit visible surface cracking after the drop in micro strain occurred. The standard specifically states this test is not accurately applicable to field practice or exposed structures. The restrained ring test is not applicable to expansive cements or concrete having a nominal maximum aggregate size (NMAS) greater than 13 mm (0.50 in.). If any of the concrete rings do not crack during the test period, the rate of tensile strength stress development at the time the test is terminated provides a basis for comparison of the materials (ASTM C 1581).

3.3.2 Effect of Curing on Shrinkage

Although multiple methods of curing are not included in the scope of this research, the method used to cure concrete is essential to its characteristics such as durability, rate of strength gain, ultimate strength, freeze/thaw resistance, and appearance. Cement paste will never completely hydrate when the w/c ratio is below 0.42. A layer of C-S-H builds up on the largest grains of cement and hinders the hydration process. Curing helps ensure as much hydration as possible occurs and at a reasonable cost (Mindess, Young, and Darwin, 2003). After meeting with the CDOT, it was discovered that training on the importance of curing techniques was non-existent, leaving a huge opportunity for project error. A survey of other state DOT’s further strengthened the widespread belief suggesting curing practices are a major cause, perhaps the primary cause, of transverse deck cracking. Krauss and Rogalla performed their own survey of existing DOT’s fifteen years ago (1993) and received many of the same responses concerning curing. They discovered many curing practices were being used in different states depending upon the job but that no standard curing practice existed for bridge decks. Practices ranged from allowing only membrane or curing compounds to requiring long-term wet curing using curing compounds, and in many cases, the contractor was given the liberty to choose the method. Krauss and Rogalla suggest the latter practice will most likely result in problems with the concrete. Typically, the contractor would choose the cheapest method to save money, but the cheapest method is not typically the most effective one for the job. Babaei and Hawkins (1987) point out that fogging or evaporation retarding films substantially reduce early plastic deck cracking if applied immediately after strike-off of the concrete. In addition, Babaei and Hawkins suggest applying wet burlap as soon as possible. This method results in fewer smaller cracks than curing compounds; delayed water curing increases cracking.

Krauss and Rogalla (1993) reported high cement content concrete to be most affected by curing. Concrete with a w/c ratio equal to 0.50 and cement content of 278 kg/m^3 (470 lb/yd^3) that was wet cured for 60 days experienced little change in time to first cracking of the ring in the restrained ring shrinkage test. When the w/c ratio was lowered to 0.35, cement content increased to 501 kg/m^3 (846 lb/yd^3), and curing remained the same, time to first cracking of the ring increased from 11.7 to 21.0 days.

Mindess, Young, and Darwin (2003) suggest the duration of and the maximum temperature reached by the cement paste plays a major role in cracking. They report pastes which achieve elevated temperatures during curing experience reduced irreversible shrinkage with no effect on reversible shrinkage. A paste exposed to 65^oC (150^oF) reduces irreversible shrinkage by 66.67% and total shrinkage by 33.33%. This reduction is attributed to the large proportion of the capillary porosity having formed as macro pores, resulting in a reduced micro porosity of C-S-H. The effective reduction in shrinkage is a function of the duration of exposure time to higher temperatures. According to Mindess, Young, and Darwin the exposure time necessary to reduce shrinkage can be relatively short and is often less than the total curing time.

Wet curing techniques such as quickly applying wet burlap, water ponding, or continuous water misting are all beneficial curing methods that reduce cracking by reducing the evaporation rate of water in concrete. High performance and high cement content concrete only have a small amount of mixture water to evaporate. Wet curing not only slows down the rate of water evaporation but cools the concrete simultaneously. This results in lower thermal stresses that develop due to the heat of hydration (Krauss and Rogalla, 1993).

Mixed opinions exist as to what is the ideal curing method. Krauss and Rogalla suggest the immediate use of windbreaks and wet curing the concrete. Curing should consist of misting, curing compound, and wet burlap. The minimum curing period is 7 days, ideally 14 days, when the evaporation rate exceeds 1 kg/m^2/hr (0.2 lb/ft^2/hr) for normal concrete and 0.5 kg/m^2/hr (0.1 lb/ft^2/hr) for concrete susceptible to early-age cracking due to low w/c ratios. They report that exposure to high temperatures after the curing period is complete can also help to reduce irreversible shrinkage. Most researchers agree that a standardized method of curing is needed and should be initiated by AASHTO.

Deshpande et al (2007) examined the effect of the curing length on air-entrained concrete made with both Type I/II and Type II coarse ground cement. Concrete made with Type I/II cement exhibited significantly increased shrinkage when comparing curing durations at different periods of time beyond initial drying. At 30 days beyond initial drying the shrinkage of concrete cured for 3, 7, 14, and 28-days were 500με (micro strain), 375, 340, and 274με, respectively. As the curing period increased, the free shrinkage decreased. This trend continued through measurements taken up to 365 days past initial drying. At 365 days past drying the largest difference in shrinkage strain occurred between concrete cured for 3 and 7 days, 690 and 515με, respectively. Differences in strain were small between concrete cured for 7 and 14 days at 525 and 500με, respectively.

Air-entrained concrete made with Type II coarse ground cement exhibited a similar trend; shrinkage decreased with increased curing periods. At 30 days past drying, concrete cured for 3, 7, 14, and 28-days experienced free shrinkage micro strains of 250, 205, 110, and 5με, respectively. Concrete cured for 3 days experienced slightly more shrinkage than concrete cured for 7 days until approximately 75 days past drying. After that point the difference in free shrinkage results were relatively small. At 180 days past drying a difference of approximately 50με existed between the concrete cured for 3 to 7 days and those cured for 14 to 28-days. It is apparent from the results that an extended curing period creates a more durable concrete for both Type I/II and Type II coarse ground cement concrete. It is clear that the ultimate shrinkage of concrete made with Type I/II cement is significantly higher than Type II coarse ground cement concrete at all ages. Free shrinkage measurements were taken at intervals of 30, 180, and 365 days past drying on concrete cured for 3 days, and a difference of 225, 240, and 300με, respectively, existed between the Type I/II and Type II coarse ground cement. The research performed by Deshpande et el (2007) clearly shows the advantage of using Type II coarse ground cement over a Type I/II cement when the effect of curing periods on shrinkage are being considered.

3.3.2.1 Internal Curing Using Lightweight Aggregate

The use of new presoaked lightweight aggregate (LWA) in high performance concrete (HCC) is becoming more common. The aggregate is said to internally cure as a result of being soaked before batching and contributes to the hydration process instead of absorbing water from the concrete mixture. This approach uses aggregate made of porous expanded shale, sufficient to provide effective internal curing in order to reduce self-desiccation and autogenous shrinkage cracking. Cusson and Hoogeveen conducted research (2006) at the Canadian Institute for Research and Construction examining high performance concrete made with Type I portland cement and partial sand replacement with LWA. A control mixture was designed with a cement-sand-stone ratio of 1:2:2 and w/cm equal to 0.34. It is noted that the water used to pre-soak the LWA was accounted for in the calculation of the w/cm and remained constant for all of the concrete mixtures examined. This requirement was said to have made the evaluation of the internal curing effectiveness more severe than if additional water had been used to soak the aggregate.

The three batches substituted normal weight sand with 6, 12, and 20% pre-soaked LWA and a fourth control mixture substituting 0% LWA. One large concrete prism 200 x 200 x 1000 mm (8 x 8 x 40 in.) was cast for each mixture with reinforcement and used a setup attaching strain gauges to the steel in order to determine the restrained shrinkage. A second concrete prism of the same size was cast from each mixture without reinforcement and used for unrestrained shrinkage testing. This prism was cast with thermal couples and relative humidity (RH) sensors (measuring self-desiccation) implanted within the fresh concrete. Compressive strength and splitting tensile strength tests were also performed on 100 x 200 mm (4 x 8 in.) cylinders. The 20% LWA concrete experienced reduced drying shrinkage due to the internal curing. The RH of the control specimen reduced from 100% at set time to 98% after 2 days and 96% after 7 days. The RH of the 20% LWA concrete reduced to 98% after 2 days and 94% at 7 days. The control test specimen had a 7 day compressive strength of 50MPa (7252 lbs/in^2) versus the 20% LWA concrete of 57MPa (8267 lbs/in^2). Cusson and Hoogeveen attribute this to the improved hydration of the pre-soaked LWA. Free shrinkage test results prove that as the LWA content increased in the concrete mixtures the autogenous shrinkage decreased. The 0, 6, 12, and 20% LWA concretes experienced strains of 252με (micro strain), 210, 112, and 46με respectively at 2 days of age. After restrained shrinkage tests were performed the stress/strength curves were normalized. This was done to compare the various curves corresponding to different concretes, which require different degrees of restraint during testing. Restraints varied from a low 0.9% for the 0% and 6% LWA concrete in order to avoid failure, to a high restraint of 1.1 for the 20% LWA concrete, having the loading system slightly pulling on the prism. The replacement of sand with LWA increased the modulus of elasticity (MOE) considerably.

At 3 to 4 days of age, the MOE was several thousand MPa higher for the 20% LWA concrete than the control (0% LWA) concrete. The 7 day splitting tensile strengths were measured to be 4.1MPa (595 lbs/in^2), 4.8MPa (696 lbs/in^2), 4.5MPa (653 lbs/in^2), and 4.2MPa (609 lbs/in^2) for the 0%, 6%, 12% and 20% LWA concretes respectively. The maximum stress/strength ratio achieved by the 20% LWA concrete was 50% after nearly 3 days. These results illustrate the LWA to be extremely beneficial in reducing cracking. Cusson and Hoogeveen's research shows how effective internal curing is against shrinkage and tensile stress in concrete, especially high performance concrete. Their results prove the effect of LWA sand replacement on strain and stress reductions. Their data indicates that a 25% LWA concrete could possibly eliminate autogenous shrinkage and tensile stress. Significant swelling did occur in the 20% LWA concrete.

As a result, it is not recommended to use more than a 25% LWA concrete because of the possibility of excess swelling (Cusson and Hoogeveen, 2006).

3.4 Design Mixture Factors Affecting Cracking in Concrete

3.4.1 Silica Fume

Substitution of cement with silica fume produces a denser concrete matrix. It results in a more rapid rate of hydration, which is accompanied by a higher heat of hydration and increased early strength development (Transportation Research Circular E-C107, 2006). A higher heat of hydration results in higher thermal stresses and reduced bleeding, making concrete more prone to plastic shrinkage (Xi et al, 2003). Another study by Bissonnette, Pierre, and Pigeon (1999) also claims silica fume is not beneficial in concrete for reducing cracking. One of their research programs compared two concrete mixtures with w/cm equal to 0.33. One of the mixtures contained 15% silica fume substitution for portland cement. Restrained ring shrinkage tests were performed and the silica fume concrete produced an additional 300 micro strains at 4 days of age over the 100% portland cement concrete. Bissonnette et al concluded that the presence of silica fume in concrete results in increased long term shrinkage. However, the resulting early age increase in shrinkage leads to significant cracking because the tensile strength is so low at early ages (1999).

Whiting, Detweiler, and Lagergren (2000) also researched the effect of silica fume on concrete shrinkage in full depth decks and concrete overlays. Full depth mixtures used lower cementitious material contents and air contents with higher w/cm than the overlay design mixtures. Silica fume substitution ranged from 0 to 12 percent of the total cementitious material weight and w/cm for overlays ranged from 0.30 to 0.35; full-depth decks w/cm ranged from 0.35 to 0.45. Unrestrained drying shrinkage tests AASHTO T 160 (ASTM 157) were performed on three 75 x 75 x 285 mm (3 x 3 x 11.25 in.) prisms molded for each mixture. The unrestrained test specimens were cured in lime saturated water; full-depth mixtures were cured for 7 days and overlay mixtures only 3 days. They were then moved to a controlled relative humidity of 50% and a temperature of 23^{o} C (73^{o} F). Restrained shrinkage tests were performed per ASTM C 1581 (AASHTO PP34) on a 75 mm (3 in.) thick, 150 mm (6 in.) high concrete ring around the outside of a 19 mm (0.75 in.) thick steel cylinder having an outside diameter of 300 mm (11.75 in.). The restrained ring specimens were cured for periods of 1 and 7 days, intending to represent both the worst and best field curing practices. A data acquisition system wired to four strain gauges that were attached (90^o offset) around the inside of the steel ring measured the strains at thirty minute time increments. Their results show the presence of silica fume to have little effect on long term shrinkage (450 days). Early age shrinkage (4 days) was higher for concrete mixtures with silica fume, versus the control mixtures made without. At this age, results consistently show an increase in shrinkage with increased silica fume content. Lower w/cm concrete mixtures (0.36) demonstrated less shrinkage when made with a constant replacement of silica fume (1.8%) than concrete with a higher w/cm (0.43). The lower the w/cm the less cement content relative to the mixture. The lower the amount of cement in the mixture results in a lower paste content for the mixture, thus a decrease in shrinkage. Whiting et al point out that the two mixtures having w/cm of 0.36 and 0.43 had paste volumes of 25.2 and 27.5%, respectively. It is also noted that small variations in w/cm may greatly influence shrinkage in concrete. The silica fume specimens cured for one day cracked earlier than the control specimens. For specimens cured 7 days, the silica fume in the concrete significantly reduced time to first crack. Whiting et al suggest not exceeding 6% silica fume replacement of portland cement because it begins to have an adverse effect on shrinkage and cracking.

3.4.2 Fly Ash

Research concerning the replacement of portland cement with fly ash in a concrete mixture has returned contradicting results. Class F and class C fly ash replacement is a very effective method of slowing the rate of C-S-H growth. It reduces early age strength gain and early concrete temperatures while achieving the same or higher ultimate strength (Xi et al, 2003). High volumes of fly ash substitution for portland cement have been studied in the past. Atis and Cabrera reported a decrease in drying shrinkage with the use of fly ash (2003). They created mixtures with varying w/cm (0.28 to 0.34) which had previously been determined to be optimal for maximum compact-ability using the vibrating slump test (Cabrera and Atis, 1999). These optimal w/cm were used in creating zero slump concrete mixtures and achieving workability by using a carboxylic type super-plasticizer. The mixtures were designed containing 100% (control mixture), 50%, and 30% portland cement replacement with a low calcium class F fly ash (ASTM C 618). Two molds of each mixture were fabricated and tested. The mixtures in the molds were the same except one used a super-plasticizer. Atis and Cabrera performed unrestrained shrinkage tests on 50 x 50 x 200 mm (2 x 2 x 8 in.) concrete prisms that had been unmolded after 24 hours and then stored at 20^{o} C (68^{o} F) and a relative humidity of 65%. Measurements were taken up to six months of age to determine changes in length (drying shrinkage) using a mechanical dial gage. The super-plasticized mixtures containing 0, 50, and 70% fly ash replacement of portland cement exhibited strains equal to 385, 263, and 294 micro strain respectively (2003). When these were compared with the same percent fly ash replacement mixtures without a super-plasticizer, the mixtures without the super-plasticizer exhibited approximately 50% less shrinkage. The compressive strengths were measured and compared between the control mixture and the fly ash concrete. The compressive strength of 50% fly ash concrete exceeded the control concrete once it reached 7 days of age. The compressive strength of the 70% fly ash concrete was exceeded by the control at all ages. The 28-day compressive strengths showed a drastic difference. The control, 50% fly ash concrete, and 70% fly ash concrete compressive strengths were 65MPa (9430 psi), 67MPa (9720 psi), and 31MPa (4500 psi) respectively. Cabrera and Atis' research suggests concrete mixtures with portland cement replacement by approximately 50 percent fly ash and no super-plasticizer to be optimum.

Research conducted at the Materials Laboratory at CU-Boulder has shown concrete made with smaller particles of fly ash certainly have some advantages over conventional concrete, but may not be applicable for bridge decks due to its high early strength, high ultimate strength, and low crack resistance (Xi et al, 2003). Some studies say both Class C and Class F fly ash replacement in concrete increases drying shrinkage and results in increased early cracking with decreased development of tensile strength (Hadidi and Saadeghvaziri, 2005). The research studied in the literature review is tough to decipher; fly ash replacement have mixed results. Its reduction in the rate of stiffness development is helpful in reducing its potential for cracking (Transportation research circular E-C107, 2006). While the reports are contradictory, the majority of the literature suggests fly ash is beneficial with regards to concrete shrinkage.

3.4.3 Water to Cementitious Materials Ratio (w/cm)

The w/cm is the ratio of the weight of the water to the weight of all cementitious materials per cubic yard of concrete. This ratio effects concrete in many ways. The permeability, porosity, ultimate strength and rate of strength gain are all affected by changes in the w/cm. It is generally accepted that drying shrinkage increases significantly as the water content increases. ACI 224 Report states that a typical concrete specimen, 134 kg/m^3 (225 lbs/yd^3) water content, resulted in a drying shrinkage of approximately 300 micro strains. In addition, it states that drying shrinkage increases at a rate of 30 micro strain per 5.9 kg/m^3 (10 lbs/yd^3) increase in water content. A study of twelve Pennsylvania bridges reported crack intensities of 0 to 87m/100m^2 (265 ft/1000ft^2) with mixture water contents varying from 158 to 173 kg/m^3 (267 to 292 lbs/yd^3). An increase in water content showed increased drying shrinkage of approximately 75 micro-strains, indicating that with respect to transverse cracking, mix water content alone was not the significant difference in the performance of bridge decks (Babaei and Purvis, 1995a). Similar articles report concrete with a w/cm greater than 0.45 tend to have high porosity and can exhibit substantial drying shrinkage, which results in reduced protection of the reinforcing steel from chlorides (Transportation research circular E-C107, 2006).

3.4.4 Cement Content

The cement content has a significant effect on shrinkage and cracking in concrete. Concrete made with higher cement content and a low w/cm is more susceptible to cracking than concrete with low cement content and higher w/cm (Xi et al, 2003). Xi et al research and other literature suggest limiting cement content to 470 lbs/yd^3 (279 kg/m^3) and that a cement paste volume less than 27.5 % can significantly reduce cracking. However, as high strength concrete has become more common in the industry, it is often encouraged to increase the cement content. Proper measures must be taken for concrete made with increased cement content or it can significantly increase cracking (Transportation research circular E-C107, 2006).

Deshpande et al (2007) conducted research using Type II coarse ground portland cement in nine concrete mixtures while varying w/cm and aggregate content. It was concluded that a clear trend for shrinkage results from variations in w/cm ratio did not exist. At 180 days of age a pattern of shrinkage occurred in the concrete having the highest aggregate content (80%). The higher the w/c the more shrinkage that occurred; 280με: w/c = 0.40 and 305με: w/c = 0.50. This wasn't the case for the mixtures containing lower aggregate contents of 60 and 70%. They reported the greatest shrinkage occurred in the concrete with a w/c equal to 0.40 (the lowest w/c) and a 60% (lowest) aggregate content. Shrinkage was lowest in the concrete with a w/c equal to 0.40 and having the highest aggregate content of 80%. The research is consistent with other literature in stating that for a given w/c ratio, the shrinkage decreases with an increase in aggregate content. The aggregate acts to restrain the concrete against shrinkage. Adversely, for a given aggregate content, the results of this study show changes in the w/cm and using coarse ground cement to have very little effect on shrinkage (Deshpande et al, 2007).

3.4.5 Cement Type

3.4.5.1 General Effects of Cement Fineness

Cement types vary depending upon the application. Different types of cement produce different temperatures as a result of their hydration processes. Some cement is ground finer than others. Further, some cement is designed for high early strength, resulting in a high heat of hydration and high thermal stresses. The resulting stresses make concrete more likely to crack. In addition, there are cement types designed to gain strength more slowly, corresponding to a lower heat of hydration (Type I/II, Type II, and Type IV). Concrete made with these cement types is expected to result in lower thermal tensile stresses and reduced cracking. Burrows (2003) reports that cracking in bridge decks increased in 1973 when the building code increased 28-day compressive strength requirements from 3000 lbs./in.^{2} to 4500 lbs./in.^{2}. The increase in the rate of strength gain causes concrete to become more brittle and likely to crack. Burrows points out that in 1966 Virginia increased its 28-day compressive strength requirements from 3000 lbs./in.^{2} to 4000 lbs./in.^{2}. It was at this time when bridge deck cracking increased from 11% to 29%. His research brings attention to numerous Colorado area bridge decks built in the 1950's that remain in great condition but are being demolished to accommodate a necessary widening of Interstate-25. The bridges of the 1950's had unacceptable 28-day compressive strengths by today's code, but have significantly maintained their structural integrity for half a century. As of 1995, Burrows reports bridge deck cracking in the United States to have increased to 52% of all bridges. This clearly illustrates the upward trend of bridge deck cracking as the required strengths and rate of strength gain continue to increase (Burrows, 2003).

Xi et al suggest using Type II cement and avoiding finely ground cement and/or Type III cement (2003). Cements with high alkali content, high C_3S and C_3A contents, low C_4AF, and high fineness have an increased development of strength and are therefore more likely to crack. This is another reason the research circular raises caution in using Type III cement for bridge decks (Transportation research circular E-C107, 2006).

3.4.5.2 Coarse-Ground Cement

Research conducted by Deshpande et al for the University of Kansas Research Center show significantly reduced shrinkage in concrete using Type II coarse-ground cement versus Type I/II cement. In addition, Deshpande examined the effect of aggregate content and w/c on shrinkage (2007). A program consisting of three concrete mixtures made with Type I/II portland cement and three mixtures made with Type II coarse-ground cement. The w/c was 0.40 for all six mixtures but the aggregate content varied between 60, 70, and 80% for each type of portland cement. At 180 days of age, free shrinkage tests measured significantly lower shrinkage strains (280$\mu\varepsilon$) in the Type II coarse-ground cement having the highest aggregate content (80%) than the shrinkage strain (665$\mu\varepsilon$) measured in the concrete made with the Type I/II portland cement having the lowest aggregate content (60%). These strains tapered off near 180 days of age and, at 365 days of age, illustrated an insignificant amount of continued shrinkage strain (Deshpande et al, 2007). This suggests that both Type II coarse ground cement and mixtures with a higher aggregate content are more suitable for use in crack resistant concrete bridge decks.

Brewer and Burrows (1951) tested three cement clinkers ground to finenesses ranging from 1200 to 2700 $cm.^2$ (186 to 419 $in.^2$), in 300 $cm.^2$ /g increments. They performed tests similar to ASTM C 1581 but using an apparatus created before the standard was adopted by ASTM. They also performed unrestrained shrinkage tests on mortar bars. Restrained shrinkage tests showed concrete made with coarse-ground cement resisted cracking longer than the more finely-ground cement concrete. They discovered the coarse-ground cement mortar bars resulted in forty percent more unrestrained shrinkage and decreased in volume at a slower rate than the more finely-ground cement concrete. It was concluded that mortars made with coarse ground cement are significantly more resistant to cracking than more finely-ground cement concrete due to drying shrinkage (Brewer and Burrows, 1951).

3.4.5.3 Shrinkage Compensating Cements

Shrinkage compensating cements (SCC) are another cement type currently being studied in the United States. Type K cement (ASTM C845-80) creates an amount of expansion when the concrete is hardening, in an effort to counteract autogenous shrinkage and drying shrinkage. ACI 223R-90 (1992) illustrates the specifications concerning the use of expansive cement. According to Xi et al, the problem with designing concrete using expansive cement is predicting the amount of expansion necessary for each individual project (2003). Krauss and Rogalla performed research using Type K cement and reported two specimens used in restrained ring shrinkage testing didn’t experience significant cracking (1996). Surface cracking occurred but no distinct cracks developed. The research shows ring strains decreased to a constant level without cracking. They also examined a SCC containing an ettringite forming additive. Control mixtures cracked at an average of 20 days and the SCC concrete time to cracking extended to approximately 36 days. Researcher’s state the restrained ring shrinkage test has merit but field performance will vary from laboratory results when SCC are utilized.

Perragaux and Brewster investigated several bridge decks for the New York State Department of Transportation in 1992. Results varied as they compared the bridge decks made using shrinkage compensating cement with surrounding structures previously made using Type II cement. In some structures it was believed that SCC reduced shrinkage by 25% and in some cases the SCC structures cracked more than the Type II cement structures. The research concluded shrinkage compensating cement is not advantageous when compared to Type II cement (Perragaux and Brewster, 1992).

Studies performed by the Ohio and New York State Departments of Transportation have returned mixed reviews concerning shrinkage compensating cement. Ohio reported success with this type of cement in bridge decks while New York had issues with durability (Philips et al, 1997). In 1989, Purvis performed research on concrete slabs made with SCC and found the final net drying shrinkage of SCC slabs was less than slabs made with Type I cement, but the SCC slabs experienced more creep.

3.4.6 Aggregate Content

Because shrinkage is a paste property, it makes sense that increasing the aggregate content decreases shrinkage. Aggregates help by providing restraint to shrinkage while occupying space within the concrete matrix; a space that would otherwise be occupied by additional cement paste. This also helps to create a more economical project because cement is the most expensive material used in producing concrete (Transportation research circular E-C107, 2006). However, aggregates themselves may be responsible for shrinkage. The use of highly absorptive aggregates has proven to result in increased shrinkage. They are more compressible and therefore allow for higher shrinkage. Some may shrink an appreciable amount themselves by the time they are completely dry (Transportation research circular E-C107, 2006).

Studies performed by Deshpande et al (2007) examined a program consisting of nine concrete mixtures made with Type II, coarse-ground portland cement. The w/c were 0.40, 0.45, and 0.50 and the aggregate contents were 60, 70, and 80%. Three mixtures were made with each w/c and aggregate contents of 60, 70 and 80%. It is clear that a delicate balance of aggregate content and w/c ratio are necessary to determine the most appropriate concrete design mixture for crack resistant bridge decks. At 180 days of age, a trend developed showing significantly less shrinkage occurring in the two concretes made with the higher aggregate content (80%) and the lower aggregate content (60%)

concrete. The 80% aggregate content concrete mixture experienced less shrinkage than the 60% aggregate content concrete. The smallest strain (280με) was produced by the highest aggregate content mixture having the lowest w/c. Accordingly, the highest strain (305με) produced was in the concrete made with an aggregate content of 80% and a w/c of 0.50 (highest w/c). This was not the case with the other six mixtures. The mixtures containing an aggregate content of 60% and 70% did not follow the same trend as the concrete made with an aggregate content equal to 80%. A 70% aggregate content produced significantly lower strain (360– 380με) than the 60% aggregate content mixtures (450–510με) (Deshpande et al, 2007).

3.4.7 Aggregate Composition

3.4.7.1 General

The type of aggregate used in concrete can affect shrinkage. Tests have shown quartzite aggregate to exhibit significantly lower shrinkage strains than concrete made using limestone aggregate. When comparing concrete made using granite, limestone, and quartzite aggregate, the shrinkage strain values at 30 days were 283, 320, and 340 micro strain respectively. These results show that granite aggregate results in the least amount of concrete shrinkage (Deshpande, Darwin, and Browning, 2007).

Xi et al states that the larger the maximum aggregate size, the smaller the resulting shrinkage. They report that when the cement paste shrinks, it cannot pull the larger surrounding aggregate closer since they are already in close proximity. Micro cracks will develop; however, as long as these micro cracks do not grow, the concrete's ability to resist cracking is enhanced and shrinkage is reduced (2003).

3.4.7.2 Aggregate Composition and Water to Cementitious Materials Content

Research conducted by Meyerson, Mokarem, and Weyers for the Virginia Department of Transportation (2003) used three types of aggregate; limestone, gravel, and diabase. Type I/II portland cement concrete mixtures with no SCMs were examined in the first three programs. Each program consisted of a range of w/c. The fourth program of mixtures were designed with cement replacements of 40% (by weight) ground granulated blast furnace slag (grade 120), Class F fly ash, and a pure amphorous micro-silica, each conforming to their appropriate standards ASTM C 989-98, ASTM C 618-97, and ASTM C 1240-97, respectively. At 7, 28, and 90 days of age compressive strength tests were performed following ASTM C 39-98 on 102 mm x 203 mm (4 x 8 inch) test cylinder specimens. Two testing programs containing 100% portland cement and w/c from 0.42-0.49 were examined. The first program had w/c of 0.49, 0.47 and 0.46 and the second program consisted of w/c of 0.45, 0.43, and 0.42 and both incorporated limestone, diabase, and gravel mixtures respectively. In Mokarem et al's research (2003) the gravel aggregate concrete mixtures consistently had the highest compressive strength, but in most cases it was not significantly stronger than the diabase aggregate. However, the limestone aggregate mixtures consistently produced significantly lower compressive strengths than both the diabase and gravel mixtures. As expected, the compressive strengths correlated to the w/c, the highest compressive strength correlating to the concrete having the lowest w/c and the lowest compressive strength correlating to the concrete having the highest w/c. These results were then tested with a third program of 100% portland cement mixtures to verify that it was not the limestone aggregate properties alone that caused a reduced compressive strength. The program included mixtures having w/c ratios of 0.33, 0.35, and 0.39 for the limestone, gravel and diabase mixtures, respectively. These are the lowest w/c of any of the programs and this is the largest variation of w/c ratios examined in Mokarem et al's research (2003). At 7, 28, and 90 days of age the limestone mixtures compressive strength significantly exceeded that of the gravel and diabase mixtures. At 7 days of age, the compressive strengths measured 7150, 6260, and 6070 lbs/in.^{2} (503, 440, and 427kg/cm.^{2}) for the limestone, gravel, and diabase mixtures, respectively. At 28 and 90 days of age the compressive strength continued to follow this trend. These results illustrate the inverse proportionality between compressive strength and w/c.

3.5 Unrestrained Shrinkage Test

Mokarem et al later performed standard tests to determine length change according to standard ASTM C 157-98. Recall, the first program of mixtures had w/c of 0.49, 0.47, and 0.46 for limestone, diabase, and gravel mixtures, respectively. The diabase aggregate concrete experienced the greatest percent length change at almost every age, although the percent changes in length between the three aggregate type mixtures were insignificant up to 56-days of age. The following unrestrained shrinkage data references programs one, two and three and the limestone, gravel, and diabase concrete mixtures, respectively. At 56-days of age, program one percent length changes were -0.0380, -0.0367, and -0.0392, program two percent length changes were -0.0342, -0.0323, and -0.0392, and program three percent length changes were -0.0321, -0.0328, and -0.0364. After 56-days of age, the diabase aggregate mixtures made with only portland cement began to experience significantly greater percent changes in length than both the limestone and gravel aggregate mixtures, while they continued to experience insignificantly different percent changes in length from one another. The results clearly show an increase in rate of length changes at later ages. At 120 days of age, program one percent length changes were -0.0431, -0.0432, and -0.0490, program two percent length changes were -0.0401, -0.0384, and -0.0457, and program three percent length changes were -0.0367, -0.0380, and -0.0453. At 180 days of age, program one percent length changes were -0.0468, -0.0462, and -0.0541, program two percent length changes were -0.0442, -0.0419, and -0.0514, and program three percent length changes were -0.0394, -0.0415, ad -0.0494. Recall the second program consisted of mixtures with w/c of 0.45, 0.43, and 0.42 (the middle range of program w/c examined) for the limestone, diabase, and gravel mixtures, respectively. When standard changes in length were measured for this program, the diabase again experienced the greatest percent length change. These tests show something interesting. The gravel and limestone mixtures experienced percent length changes correlating to their w/c ratio. The limestone concrete having a w/c of 0.45 experienced a greater percent length change than the gravel concrete having a w/c of 0.42. These results support the idea that a higher w/c equates to more water in the mixture and therefore, more shrinkage. However, the diabase concrete had a w/c (0.43) in the middle of the three mixtures and yet it experienced a significantly larger percent length change. When the third program having the lowest range of w/c was examined, the unrestrained shrinkage results illustrate a trend correlating the highest w/c (diabase, 0.39) to the largest percent length change, and the lowest w/c (limestone, 0.33) to the smallest percent length change. Mokarem et al attribute this to the to the diabase aggregate absorption value of 1.04%, versus the limestone and gravel aggregate which had absorption values of 0.48% and 0.75% respectively. These values indicate that the diabase has more voids filled with water than the other aggregate, which can increase drying shrinkage (2003).

When comparing the SCM mixtures, researchers examined mixtures containing the same type of diabase aggregate and the same w/c ratio. The mixtures containing fly ash experienced the greatest shrinkage. Micro silica and Ground Granulated Blast Furnace Slag (GGBFS) mixtures were insignificantly different from one another. The drying shrinkage in the mixtures containing SCM's exceeded that of the 100% portland cement mixtures being compared against. This is possibly due to the denser concrete matrix created when using SCM's. Capillary voids are smaller and would exude less water than normal, larger capillary voids according to Mokarem et al. This is where drying shrinkage primarily occurs (Mokarem, et. al., 2003).

3.6 AASHTO PP34 / ASTM C 1581

In the ASTM C 1581 (AASHTO PP34), Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage, a concrete ring is cast around a steel ring. Before it was adopted as a standard by ASTM, dimensions of both the steel and concrete ring for the test were modified for various reasons. The current standard (AASHTO PP34, ASTM C 1581) specifies the steel ring to have a wall thickness of 0.50 +/- 0.05 in. (13 +/- 0.12 mm), an outside diameter of 13.0 +/- 0.12 in (330.0 +/- 3.3 mm), and a height of 6.0 +/- 0.25 in. (152.0 +/- 6.0 mm), machined smooth on all surfaces. The concrete ring molded around the steel ring is 1.50 in. (38.0 mm) thick. The specimens must be transferred to the testing environment within ten minutes of completion of casting. Four strain gauges are mounted at mid-height (offset 90^o) around the inside of the steel ring. A data logger begins recording strain measurements within two minutes of the rings being placed in the testing environment. As the concrete ring experiences shrinkage (volume decrease), stresses develop resulting from the steel ring restraining the concrete. The time and micro strain is recorded upon start and micro strain values are recorded by a data acquisition at intervals not to exceed 30 minutes. Moist curing of the molds must begin within 5 minutes of the first strain reading. Moist curing continues for twenty-four hours using wet burlap, a relative humidity of 50% +/- 4%, and at a temperature of 73^o +/- 3.5^o F. Micro strain averages are recorded at pre-determined days of age and cracking is recorded to the nearest 0.25 day. When cracks occurs the most recently recorded micro strain prior to cracking is examined. This reading is used as a basis for equations which estimate the micro strain at the actual time of cracking.

Over time, variations of the ring test have been performed. The dimensions of the rings used for the test were altered several times. Krauss and Rogalla (1996) examined the effect of changing the dimensions of the rings used for the test. They placed shrinkage stresses that were both uniform and increasingly linear stress from the interface between the concrete and the steel, on the steel ring. They expected this to represent circumferential surface drying or drying from either the top or bottom surface. The research discovered that the height of the steel rings affected the shrinkage stresses in the concrete. As the height increased from 76 mm (3.0 in.) to 152 mm (6.0 in.), shrinkage stresses were reduced. Krauss and Rogalla varied the ring thickness from 13.0 mm (0.50 in.) to 25 mm (1.0 in.) but found little difference in the shrinkage stresses or cracking tendency. Thinner steel rings were associated with higher stresses in the steel and the stresses in the concrete rings increased as the steel ring thickness increased (1996).

Attiogbe et al examined ring data involving the thickness of the concrete ring versus it's time to cracking. They discovered that the concrete ring thickness was linearly proportional to its time to cracking and that the depth of drying increases proportionally with the square root of drying time (2004). ASTM C 1581 (AASHTO PP34) is regarded by the engineering field to be a valid and extremely valuable standardized test to determine the durability of concrete, especially when considering concrete cracking in bridge decks.

3.6.1 Restrained Ring Shrinkage Test

Mokarem et al performed the restrained ring shrinkage testing (AASHTO PP34-98) for a period of 180 days of age, on 42 ring specimens, and strain measurements recorded at 7, 28, 56, 90, 120, 150, and 180 days of age (Mokarem et al, 2003). Average strains were calculated at each of these days and equations based upon the most recent strain record prior to cracking were used to estimate the strain at any day. In the first program of mixtures, the diabase rings never cracked through the end of the test period. At the end of 180 days, the diabase concrete rings had an average micro strain value of -132με, significantly less than the limestone and gravel concrete rings. The limestone and gravel concrete rings cracked at 125 days and 117 days, respectively. At cracking, the limestone ring was determined to have an average micro strain value of -234με at approximately 120 days, meaning it cracked when it reached a value slightly higher than -234με. When the gravel concrete ring cracked, it had an estimated micro strain value of -210με. Mokarem et al report the diabase aggregate concrete had a lower modulus of elasticity (MOE) than the limestone and gravel concrete and researchers believe this may have been why the diabase concrete didn't crack. Mokarem et al state that a higher modulus of elasticity concrete is stiffer and possibly able to resist shrinkage in an unrestrained condition, but the stiffer concrete may create higher strains on the ring in a restrained condition. The mixtures from program two didn't crack. At 180 days, the average micro strain values for the limestone, gravel, and diabase concrete were -168, -194, and -200με, respectively. Again, the modulus of elasticity is possibly the cause for the trend in micro strain. The concrete associated lowest MOE having the restrained shrinkage strains. The third program had the lowest of the w/c for the limestone, gravel, and diabase concrete mixtures. Only the gravel and diabase experienced cracking at 165 and 172 days, respectively. The diabase and gravel concrete rings both had an estimated micro strain value of -210με at cracking. Researchers attribute this program's trend in cracking to the w/c. Lower w/c should theoretically experience less shrinkage. In the third program the w/c ratio was the lowest for the limestone, which experienced a significantly lower amount of strain than the diabase and gravel concrete. None of the rings from the fourth program cracked during the 180 day test period. These mixtures contained SCM's and experienced lowest strains of any programs mixtures. Average micro strain values ranged from -142με to -193με for all of the mixtures at 180 days of age. Mokarem et al note that the strain measured for the fly ash concrete was the highest for both the restrained and unrestrained shrinkage tests. The slag concrete measured the lowest average micro strain value at the end of the test period. Researchers looked at data for the four rings that broke and each ring had a micro strain greater than -200με. Therefore, it was estimated that micro strains greater than -200με will result in cracking of restrained drying shrinkage rings. Using data obtained from concrete having an average micro strain value of -200με, it was determined that a strong correlation existed.

3.7 Length Change

The corresponding length change associated with concrete having restrained shrinkage strain measuring -200με was thought of as a standard. A percent length change that exceeded those resulting from -200με were then said to increase the probability of cracking. In Mokarem et al's research, linear equations for each mixture group were used in calculating associated percentage length changes. Percent length changes in excess of -0.0342, -0.0478, and -0.0482 were determined to correlate with the cracking of the 100% portland cement mixtures in programs one, two, and three respectively. The mixtures containing SCM's would likely crack if percent length changes occur in excess of -0.0516. Mokarem et al concluded that for 100% portland cement mixtures, 28-day percent length change should be limited to -0.0300 and -0.0400 at 90 days to reduce the risk of cracking due to drying shrinkage. For SCM concrete, percent length change should be limited to -0.0400 at 28-Days and -0.0500 at 90 days.

3.8 Admixtures

Water reducing admixtures are often used in concrete to increase workability while maintaining a low w/cm, resulting in higher concrete strength. A lower w/cm will result in reduced drying and plastic shrinkage.

ACI 212 Committee Report (ACI 212, 1989) gives detailed information concerning set retarders and set accelerators. Set retarders are sometimes used in bridge deck applications because they offer delayed set times. These retarders allow for continuous placement of bridge decks making the deck less susceptible to cracking due to deflection of the formwork during placement. The delayed set time is also accompanied by lower temperatures during hydration which help reduce cracking due to thermal stresses (Transportation research circular E-C107, 2006).

Xi et al state that there is no definite conclusion on the influence of set controlling admixtures on bridge deck cracking. The use of retarders increases plastic shrinkage, but decreases the heat of hydration and thermal stresses, resulting in decreased drying shrinkage cracking (2003).

Shrinkage reducing admixtures (SRAs) are a new product currently undergoing testing and research. They work by reducing the surface tension of the concrete water which reduces internal stresses thus lowering long-term shrinkage. Concrete in the 50% humidity range develop significant capillary stresses which develop into cracks. SRAs reduce these stresses enough to reduce shrinkage cracking. There has been a significant amount of research on SRAs included in laboratory trials; however, limited research was found in which SRAs were incorporated in bridge decks.

CHAPTER 4 - PROBLEM STATEMENT

4.1 Statement

If concrete cracks during the early stages after placement, it immediately begins to degrade the structure. Preventing the early age cracking of concrete is especially important to the CDOT. It is the CDOT's responsibility to maintain a safe network of roads, bridges, and highways throughout the State of Colorado. From public safety to keeping an efficient budget, a durable low cracking potential concrete is very effective in accomplishing both of these objectives. A cracked bridge deck not only diminishes the integrity of the structure but jeopardizes the safety of the travelling public. Substantial damage to the structures integrity begins to occur when cracking in the deck surface allows water to penetrate to the reinforcing steel. The resulting corrosion of steel reinforcement shortens the life span of the bridge and increases maintenance costs while the bridge is in service. These factors are unfavorable, specifically to the department of transportation.

Winter conditions in Colorado create the need for increased deicing salt on the road surface to ensure the safety of the traveling public. The increased amounts of deicing chemicals accelerate the corrosion process when melting snow transports the chlorides through the small cracks to the steel reinforcement.

Research has been underway to investigate several factors contributing to the problems surrounding early age cracking in concrete. The CDOT currently has specifications for low cracking concrete used for bridge decks; Class H and Class HT concrete. Current specifications require fresh and hardened concrete properties of the concrete to fall within a specific range. While the current Class H and HT specifications are an improvement over previously designed bridge deck concrete, the need for enhancement still exists.

The purpose of this research is to design mixtures with material content ranges above and below that of the current specifications. It is believed that the current specifications are creating favorable scenarios for early age cracking. The rate of strength gain, magnitude of ultimate strength, permeability, restrained shrinkage strain, and freeze/thaw durability were tested for each of the designed mixtures and their effects on early age cracking examined. Specifically, eleven, low cracking potential, concrete mixtures were designed, batched, and tested for this study. Fresh and hardened concrete properties were examined and their individual effect on concrete cracking analyzed.

The primary benefit gained from this research is that the CDOT will be in a better position to design and construct crack resistant bridge decks and other concrete structures. Results from this study will provide the necessary information to develop more durable concrete bridge decks. This data will allow the CDOT to make changes to current specifications for future construction.

Ancillary benefits from this research will include a cost savings to the CDOT. Developing a crack-resistant concrete will benefit the CDOT by providing for longer lasting concrete structures and reducing the annual costs to maintain these pavement structures.

CHAPTER 5 - STATE DOT SURVEY

5.1 General

A national survey of state Departments of Transportation was conducted with the objective of obtaining additional information that may aid in the improvement of the current CDOT specification for structural bridge deck concrete. A web-based tool called SurveyMonkey.com (http://www.surveymonkey.com/) was used to formulate the questionnaire and analyze the responses. A 38% response rate was obtained for the State DOT survey. Though the response rate was not as high as the study team had hoped, valuable information was gathered from the survey findings. The survey was submitted to state Departments of Transportation (DOT) Materials and Bridge engineers. Analysis was performed on the results and aided in the concrete mixture design process of this study.

5.1.1 Survey Response

Responses were received from 19 of the 50 State DOT's, for a 38% return rate. See Figure 5.1. Multiple state DOT's provided more than one response. Most of the two-respondent states included responses from both the Materials and Bridge Engineer. The survey returned a total of 33 responses; however, only 28 individuals completed the survey.

Multiple responses were obtained from six states: Maryland Transportation Authority, Michigan Department of Transportation, Louisiana Department of Transportation, Tennessee Department of Transportation, Nebraska Department of Roads, and the Arkansas State Highway and Transportation Department.

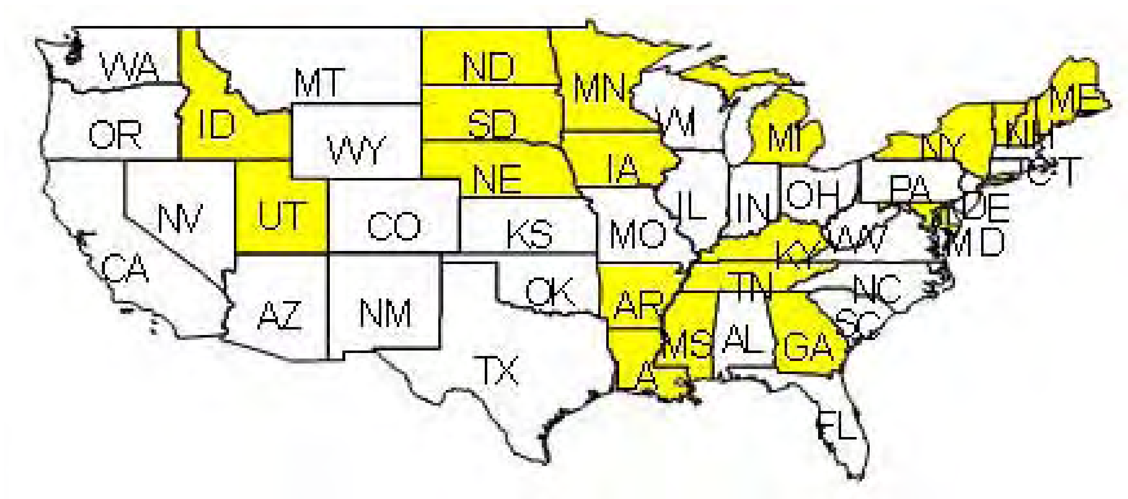

Figure 5.1 DOT Respondents Map

5.1.2 State DOT Bridge Deck Cracking Problem

A majority of respondents, 95.0%, replied that their state does experience bridge deck cracking. Transverse deck, full width cracking is common and is expected to occur at early ages in many states. In addition, the span type (i.e. continuous spans) with positive and negative moment regions have affected the frequency of cracking.

5.1.3 Potential Causes for Bridge Deck Cracking

The Respondents were asked to choose which of the following choices primarily contributes to bridge deck cracking; placement, curing, rate of strength gain, mixture design, or the use of admixtures. The majority of responses selected curing to be the primary cause of cracking. After curing, mixture design, placement, rate of strength gain, and use of admixtures were ranked most to least influential, respectively. Settlement and early-age thermal cracking were also mentioned as causes for deck cracking.

5.1.4 Rate of Concrete Strength Gain

The Respondents were asked to select at what age their bridge deck concrete typically reaches its ultimate strength; 3, 7, 14, 21, 28, or 56 days. A majority of states, 42.9%, reported achieving ultimate strength at 7 days. Respondents representing 35.7% claim to achieve ultimate strength at 28 days. Of the fourteen responses, no one reported achieving ultimate strength at 3 days of age. The information suggests that it would be beneficial to slow the rate of strength gain for the concrete being designed for this study.

5.1.5 AASHTO PP34 Ring Test Usage by State DOTs

A majority of Respondents, 93.8%, replied that their state does not perform AASHTO PP34. Many agree that shrinkage is an important issue contributing to cracking; however do not perform any shrinkage measuring tests. One response reported using the test, but finding little increased strain and zero cracking.

5.1.6 Mixture Design Issues

The respondents had to choose from four choices pertaining to mixture design; water to cementitious material ratio (w/cm), cement content, chemical admixtures, or pozzolans. Half of the respondents report cement content as the major contributor to bridge deck cracking, while 37.5% report the cause to be the water to cementitious material ratio. Pozzolans were selected only two times and chemical admixtures were not selected by anyone taking the survey.

5.1.7 Mixture Design Modifications Used to Improve Concrete Performance

A common adjustment made by many states is the cement content. Reductions in cement content were mentioned; 660lb/yd^3 to 611lb/yd^3, and 709lb/yd^3 to 571lb/yd^3. An approach taken by the Minnesota DOT is to reduce the permeability of the concrete with lower paste contents and higher percentages of SCM's. However, their latest designs involve straight portland cement. The concern of the Minnesota DOT is that the use of supplementary cementitious materials (SCM's) result in lower tensile strengths in the first several days resulting in concrete unable to resist restraint cracking.

5.1.8 Shrinkage-Reducing Admixtures

Only 21.4% of the responses indicated using shrinkage-reducing admixtures in their states bridge deck concrete. The Michigan DOT abandoned a project involving SRAs claiming it repeatedly “knocked the air out.” An ongoing project currently utilizing

SRAs is the Twin Spans Bridge between New Orleans and Slidell. Because the project is ongoing, LADOT has not yet reported whether it was or was not beneficial.

5.1.9 Shrinkage Compensating Cement

Shrinkage compensating cement (Type K, expansive cement) is currently being tested in the United States. Ohio and New York are two of only several states currently utilizing this type of cement. One problem concerning Type K cement is predicting the amount of expansion that will occur. The majority of the respondents, 84.6%, reported having never used shrinkage-compensating cement in their bridge decks.

5.1.10 Factors Affecting Cracking (Mixture Design)

Admixtures may contribute to bridge deck cracking. Seven choices were provided for selection as materials commonly found in bridge deck concrete mixtures. The choices were silica fume, Class C fly ash, Class F fly ash, blast-furnace slag, water-reducing admixtures (super-plasticizers), set retarders, or shrinkage-reducing admixtures. Silica fume was chosen by most respondents as the cause of increased cracking. Blast furnace slag and water reducing admixtures were also selected numerous times. Louisiana suspects they are having problems with the compatibility of materials such as cement, admixtures, and fly ash within their mixture. Set retarders and shrinkage-reducing admixtures were chosen least among the provided choices.

5.1.11 Beneficial Factors that Reduce Concrete Cracking

Contrary to the responses presented in 5.1.10, some responses claim that blast furnace slag and water-reducing admixtures proved beneficial in reducing cracking in bridge decks. Some states also claim silica fume to be beneficial against cracking. The Iowa DOT reported having lower shrinkage when slag and Class C fly ash were used as a ternary blend.

5.1.12 Water-to-Cementitious Materials Ratio

Four ranges of w/cm ratio were provided for this question; w/cm < 0.35, 0.35 < w/cm < 0.40, 0.40 < w/cm < 0.45, and w/cm > 0.45. A majority of respondents, 78.6%, selected

$0.40 < w/cm < 0.45$ as the range for the maximum allowable w/cm for their State DOT's concrete bridge deck mixtures.

5.1.13 Curing Practices

Curing is mentioned by many respondents to contribute significantly to bridge deck cracking. A significant number of respondents, 81.8%, reported changes in their state's curing practices of bridge deck concrete. A common response was that an increase in moist-cure (wet cure) times from 7 to 14 days was beneficial. Another is the application of wet burlap within 30 minutes after placement. The Michigan DOT specifies strict fogging, burlap, soaker hose systems for a continuous 7 day wet cure, but reports that enforcement of these specifications is inconsistent. In addition, it was noted that monitoring concrete temperature and protection of the concrete during its early plastic state are essential in minimizing concrete cracking.

5.1.14 DOT Survey Conclusion

The results from this survey were utilized when designing concrete mixtures for this study. The information was used by the University of Colorado Denver research team in conjunction with the CDOT. The survey was successful in finding a solid foundation of information from which to begin designing concrete mixtures. In addition, it should be noted that bridge deck cracking is not an isolated phenomenon in Colorado, rather is experienced in most all states.

In summary, several factors such as cement content and concrete curing were noted as being influential factors resulting in concrete cracking of bridge decks for several DOTs. Reduction in the total cementitious content and 14 day cure times are a few adjustments to the mixture design and curing practices made by State DOTs. Further, many DOTs do not perform shrinkage evaluation tests of any kind on their current bridge deck mixtures.

CHAPTER 6 - EXPERIMENTAL DESIGN

6.1 Design Plan

6.1.1 Literature Review

A primary objective of this research includes providing the CDOT with an up-to-date investigation into crack resistant concrete for Colorado bridge decks. This involved extensive use of the internet to find applicable information about pertinent previous and current research. The review also included close examination of several published theses from various universities, students, and engineers around the world. This information was used in the design process of the eleven concrete design mixtures tested during this research.

6.1.2 Mixture Design Process

Eleven concrete mixtures were designed, batched, and tested for study. In addition to the DOT survey and literature review, design input was gathered from meetings with CDOT engineers and other industry professionals interested in the research.

6.1.3 Mixture Designs

Ultimately, eleven mixtures were developed and tested during this research study. See Table 6.1. Four mixtures were designed to reduce the early age accelerated strength gain by limiting the 7-day compressive strength to 3000 psi. This was accomplished by adjusting the w/cm, cementitious content of the mixture, and percent of pozzolan replacement. In addition, the use of coarse-ground cement was incorporated into several mixture designs.

Table 6.1 Mixture Design Matrix

Mix #	Mixture ID	w/cm	Cementitious Content	Type of Cement	%FA	%BFS	%SF	ADMIX.	Air Content (%)	Paste Vol.
1	0.38/6.8/FA20/SF5/II	0.38	640	Type II	20		5		6.5	0.28
2	0.42/6.2/FA16/SF3.5/II	0.42	580	Type II	16		3.5		6.5	0.26
3	0.38/6.8/FA20/SF5/G	0.38	640	Class G Oil Well Cement (Coarse Grained Cement)	20		5		6.5	0.28
4	0.42/6.2/FA16/SF3.5/G	0.42	580	Class G Oil Well Cement (Coarse Grained Cement)	16		3.5		6.5	0.26
5	0.44/6.5/FA30/II	0.44	611	Type II	30				6.5	0.29
6	0.44/6.5/FA30/SF5/II	0.44	611	Type II	30		5		6.5	0.29
7	0.44/6.5/BFS50/II	0.44	611	Type II		50			6.5	0.28
8	0.44/6.5/FA30/RET/II	0.44	611	Type II	30			RET.	6.5	0.28
9	0.44/6.5/FA30/SRA/II	0.44	611	Type II	30			SRA.	6.5	0.28
10	0.42/6.0/II(LWA)	0.42	564	Type II					6.5	0.25
11	0.42/6.0/II(NORM.WT.)	0.42	564	Type II					6.5	0.25

Key: 0.38/6.8/FA20/SF5/II

w/cm

Cement Content (sacks)

% Fly Ash

% Silica Fume

Type of Cement

Within the eleven concrete mixture designs are two Class H control mixtures, per current CDOT Structural Concrete Specifications. One mixture contains the highest allowable percentage replacement of portland cement with fly ash and silica fume (and lowest allowable w/cm) and the other with the lowest allowable percentage replacement of cement with the same (and highest allowable w/cm). All of the mixtures take into account aggregate content, effective replacement percentages of portland cement with supplementary cementitious materials, chemical admixtures, and varying w/cm. An air-entraining agent (AEA) was used to increase durability of the concrete. Air content within these concrete mixtures was expected to coincide with the required percentages per CDOT structural concrete specifications.

6.1.3.1 Cement Type

Mixtures #1 (0.38-6.8-FA20-SF5-II) and #3 (0.38-6.8-FA20-SF5-G) are CDOT control mixtures and have identical mixture proportions and w/cm equal to 0.38; however, Mixture #3 is made using the Type G, oil-well cement which is more coarsely ground than common Type II cement.

Mixture #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/G) are the other CDOT control mixtures but Mixture #4 is again made using the Type G, oil-well cement which is more coarsely ground instead of more common Type II cement.

6.1.3.2 Supplementary Cementitious Materials

Mixture #5 (0.44/6.5/FA30/II), Mixture #6 (0.44/6.5/FA30/SF5/II), and Mixture #7 (0.44/6.5/BFS50/II) have the same w/cm (0.44) but each introduces various amounts of cement replacement with supplementary cementitious materials; 30% Class F fly ash alone, 30% Class F fly ash and 5% silica fume, and a mixture containing only 50% blast furnace slag. The 30% replacement of cement with Class F fly ash in Mixture #5 exceeds the current allowable CDOT Class H and HT specification replacement percentage of 20%.

6.1.3.3 Chemical Admixtures

Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II) are identical in mixture proportions but each incorporates the use of a chemical admixture. Both mixtures exceed current allowable CDOT Class H and HT specification replacement percentages by having a 30% percent replacement of cement with Class F fly ash. Mixture #8 (0.44-6.0-FA30-SRA-II) utilizes a SRA to help reduce and control the development of shrinkage strain. SRAs are used in the field to help control shrinkage strain development. The SRA used in this research. was the Master Builders- Tetraguard and the maximum suggested dosage rate of 1.5gal/yd.^{3} was incorporated. Chemical properties for the shrinkage reducing admixture are provided in Appendix B. Mixture #9 (0.44-6.0-FA30-RET-II) utilizes a set retarder admixture. These admixtures are often used in the field to delay set time when temperatures are high or traffic delays delivery of fresh concrete. The set retarder was a Master Builders- Pozzolith 100XR and an average dosage of 3 ounces per one hundred pounds of cementitious materials in the mixture. Chemical properties for the Pozzolith 100XR can be found in Appendix B.

6.1.3.4 Aggregate Type

Mixture #10 (0.42-6.0-II-L.W.A) is a 100% portland cement mixture made with a substitution of normal weight sand with 250lbs./yd.^{3} of lightweight, fine-aggregate. The aggregate was pre-conditioned (pre-soaked) to a moisture content (MC.) of approximately 18%. This was an exceptionally high MC for aggregate but is done so with the intent of internally cure the concrete. The aggregate releases internal water for use in hydration of cement particles over time. Results were expected to be most significant at 56-days of age. Mixture #11 (0.42-6.0-II-Norm.Wt.) was a control mixture for comparison with the lightweight aggregate concrete mixture. Mixture proportions are identical to Mixture #10 (0.42-6.0-II-L.W.A).

6.2 Acquisition of Raw Materials

6.2.1 Cement

Two types of cement were used in this research study. Colorado produced Holcim Type II portland cement was supplied by Holcim Inc. and used in the fabrication of several concrete mixtures. In addition, coarse-grained cement supplied by GCC Dacotah Cement from Rapid City, South Dakota, was utilized for two mixtures. This type of cement is a Class G, Oil-well cement. Calcium silicate compounds and other calcium compounds containing iron and aluminum make up the majority of this product. It was expected that concrete mixtures containing this cement develop strength much slower than mixtures containing the Type II cement promoting less shrinkage and more resistance to cracking. The cement reports supplied by the manufacturers for the Holcim Type II and Dacotah Class G Oil-well cement are included in Appendix B. However, the cement compounds, chemical and physical properties and compressive strength properties for the Class G Oilwell cement are shown in Tables 6.2, 6.3, and 6.4 respectively.

Table 6.2 Class G Oilwell Cement Compounds

Dacotah Cement Major Compounds:	
$3CaO.SiO_2$	Tricalcium silicate
$2CaO.SiO_2$	Dicalcium silicate
$3CaO.Al_2O_3$	Tricalcium aluminate
$4CaO.Al_2O_3.Fe_2O_3$	Tetracalcium aluminoferrite
$CaSO_4.2H_2O$	Calcium sulfate dehydrate (Gypsum)

Table 6.3 Class G Oilwell Cement Chemical and Physical Properties

Chemical	Physical			
MgO (%)	-	-	1.2	-
SO_3 (%)	-	-	2.2	-
Ignition Loss (%)	-	0.8	-	-
Equivalent alkalies (%)	0.21	-	-	-
Insoluble residue (%)	-	0.29	-	-
C_3S	-	-	-	54
C_3A	-	-	-	4

Blaine Fineness (m^2/kg)	325
Percent Passing No. 325 Mesh, %	84
Free Water, ml	1.4

Table 6.4 Class G Oilwell Cement Compressive Strength Properties

Compressive Strength	8 hours, 100 degree F. at Atm. Press., MPa (psi)	N/A
	8 hours, 104 degree F. at Atm. Press., MPa (psi)	11.1 (1613)
Pressure Temperature Thickening Time Test	Thickening Time, minutes	131

Chemical and physical properties and compressive strength properties for the Holcim Type II cement are shown in Tables 6.5 and 6.6 respectively.

Table 6.5 Holcim Type II Cement Chemical and Physical Properties

Chemical	Physical			
MgO (%)	-	-	1.2	-
SO_3 (%)	-	-	3.2	-
Ignition Loss (%)	-	2.4	-	-
Equivalent alkalies (%)	0.7	-	-	-
Insoluble residue (%)	-	0.53	-	-
C_3S	-	-	-	56
C_3A	-	-	-	6

Blaine Fineness (m^2/kg)	396

Table 6.6 Holcim Type II Cement Compressive Strength Properties

Compressive Strength	3 Day	28.7 (4170)
	7 Day	37.0 (5360)
Pressure Temperature Thickening Time Test	Thickening Time, minutes	137

6.2.2 Aggregate

Coarse and fine aggregate were obtained from representative sources within Colorado. The UCD Materials Testing Laboratory acquired both the coarse and fine aggregate conforming to the ASTM C33 standard. Bestway Aggregate provided material properties and gradation reports for the aggregate. The aggregate properties and gradation have been checked and verified to meet Class H and HT concrete specifications.

The coarse aggregate meets the ASTM C33 Size Number 57 and 67 gradation requirements. The coarse aggregate was obtained from a source located in Brighton, CO. The fine aggregate meets the ASTM C33 gradation requirement for concrete fine aggregate. Based upon laboratory tests performed by WesTest of Denver, Colorado, this aggregate has a low potential for deleterious alkali-silica behavior. The material properties data for both coarse and fine aggregate are included in Appendix B.

The lightweight aggregate utilized in Mixture #10 (0.42-6.0-II-L.W.A) was obtained by Texas Industries Inc. (TXI). The material properties for this aggregate were provided by the supplier.

6.2.3 Admixtures

Chemical admixtures used for water-reducing (workability) and air-entrainment, as well as shrinkage reduction and set time were utilized in the design mixtures for this research.

6.2.3.1 High-Range Water Reducing Admixture (H.R.W.R.A.)

A CDOT approved high range water reducing admixture was incorporated into several of the design mixtures. The admixture was manufactured by W.R. Grace- Daracem 19, ASTM C494 Type A and F, and ASTM C1017 Type I. Chemical Properties for the Daracem 19 is provided in Appendix B.

6.2.3.2 Air-Entraining Agent (A.E.A.)

A CDOT approved AEA was utilized for the purposes of air-entraining the concrete mixtures made for this research. The agent was made by W.R.Grace- Daravair_AT60, ASTM C 260. Chemical properties for the Daravair- AT60 are provided in Appendix B.

6.2.3.3 Shrinkage-Reducing Admixture (S.R.A.)

A CDOT approved shrinkage-reducing admixture was utilized for the purposes of this research. The admixture was supplied by BASF- Master Builders_Tetraguard_AS20. Tetraguard_AS20 product data sheets are included in Appendix B

6.2.3.4 Set Retarder (RET)

A set retarding admixture was utilized for the purposes of this research. The admixture was manufactured by BASF- Master Builders_Pozzolith_100XR. Pozzolith_100XR product data sheets are provided in Appendix B.

6.3 Testing

The mixtures were tested according to ASTM standards for different characteristics occurring from 1 day of age through 56-days of age and beyond. The batching followed ASTM C 192 Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory (AASHTO T 126-97 Making and Curing Concrete Test Specimens in the Laboratory). Both fresh and hardened concrete properties were examined for each mixture batched. The fresh concrete properties that were examined include slump (ASTM C 143, AASHTO T 119), unit weight (ASTM C 138, AASHTO T 121), air content (ASTM C 231, AASHTO T 152), and concrete temperature (ASTM C 1064, AASHTO T 309). Hardened concrete properties that were evaluated in this research included compressive strength (ASTM C 39, AASHTO T 22), restrained ring shrinkage testing (ASTM C 1581, AASHTO PP 34), freeze/thaw durability (ASTM C666, Procedure A, AASHTO 161), and rapid chloride ion penetrability (ASTM C 1202, AASHTO T 227).

In addition to the durability, strength, and permeability testing of the mixtures, the shrinkage strain within the concrete was the primary focus of this research. Throughout the life of the concrete, shrinkage strain results from internal stresses created from the depletion of water. As concrete ages, water is continuously depleted by both the exposed surface evaporation of water and the continuous hydration of the internal cement particles. Restrained ring shrinkage testing allowed for an investigation into the development of allowable strain/stress versus time for each mixture before the concrete cracks. A summary table of test procedures is shown in Table 6.7.

Table 6.7 Fresh and Hardened Concrete Properties Tests

Fresh Concrete Tests	Standard	Time of Test
Slump	ASTM C 143, AASHTO T 119	At Batching
Unit Weight	ASTM C 138, AASHTO T 121	At Batching
Air Content	ASTM C 231, AASHTO T 152	At Batching
Temperature	ASTM C 1064, AASHTO T 309	At Batching
Hardened Concrete Tests		
Compressive Strength	ASTM C 39, AASHTO T 22	1, 3, 7, 28, 56 Days
Rapid Chloride Ion Penetrability	ASTM C 1202, AASHTO T 227	28, 56 Days
Durability (F/T Resistance)	ASTM C 666, Procedure A AASHTO 161	28 and Subsequent Days
Restrained Shrinkage	ASTM C 1581, AASHTO PP34	Until Cracking

6.4 Data Analysis

Resulting test data collected from this research was compared and used to provide recommendations for modifications to the current Class H and HT specification, thereby producing a more crack resistant concrete for use as bridge decks by the CDOT.

CHAPTER 7 - EXPERIMENTAL RESULTS

7.1 Overview

A total of eleven mixtures were designed, batched, and tested to develop recommendations for a crack resistant concrete. Cementitious content, cement type, water-to-cementitious ratio, pozzolan content, chemical admixtures, aggregate type, and paste content were all examined in this study. Fresh and hardened concrete properties were tested for each mixture and comparisons were made to develop conclusions regarding the effect of each examination on the cracking potential of concrete.

7.2 Fresh Concrete Properties

Fresh concrete tests included temperature, air content, unit weight, and slump. Fresh concrete properties for the eleven mixtures are listed in Table 7.1.

Table 7.1 Fresh Concrete Properties

Mixture Identification	Slump	Air Content	Unit Weight	Ambient Temperature	Concrete Temperature
	(in.)	(%)	(lbs./ft.^{3})	(^{o}F)	(^{o}F)
0.38/6.8/FA20/SF5/II	3.0	5.5	142.4	59	58
0.42/6.2/FA16/SF3.5/II	4.5	8.0	134.2	56	58
0.38/6.8/FA20/SF5/G	3.5	3.4	147.8	59	62
0.42/6.2/FA16/SF3.5/G	5.0	9.5	137.2	62	60
0.44/6.5/FA30/II	8.0	4.5	143.8	62	59
0.44/6.5/FA30/SF5/II	6.5	9.0	135.8	72	69
0.44/6.5/BFS50/II	3.5	3.5	146.4	72	68
0.44-6.0-FA30-SRA-II	3.0	2.8	147.4	74	71
0.44-6.0-FA30-RET-II	3.0	7.5	141.4	72	71
0.42-6.0-II (L.W.A)	2.5	7.5	138.6	72	72
0.42-6.0-II (Normal Wt.)	2.0	7.5	143.0	66	69

7.2.1 Slump

Current Class H and HT specifications do not specify a slump value. For adequate workability the desired slump was 3.5 inches (8.89 cm). Although some values fall below the target, all eleven design mixtures achieved sufficient workability to form test specimens. The use of a High Range Water Reducing Admixture (HRWRA) and Air Entraining Admixture (AEA) was required to obtain the needed workability and durability sought for this research.

7.2.1.1 Cement Type

Mixture #1 (0.38-6.8-FA20-SF5-II) vs. Mixture #3 (0.38/6.8/FA20/SF5/G), and Mixture #2 (0.42-6.2-FA16-SF3.5-II) vs. Mixture #4 (0.42-6.2-FA16-SF3.5-G) are CDOT Class H control mixtures examining the effect of coarse-ground cement versus the specified Type II cement. When comparing the slump values between the mixtures made using Type G, coarse-ground cement and Type II cement, the coarse ground cement concrete mixtures achieved an increased slump average of 0.5 inch (1.27cm) over the Type II cement concrete mixtures.

Mixtures #2 (0.42-6.2-FA16-SF3.5-II) and #4 (0.42-6.2-FA16-SF3.5-G) have a w/cm equal to 0.42 and required less HRWRA than Mixtures #1 (0.38-6.8-FA20-SF5-II) and #3 (0.38/6.8/FA20/SF5/G), which both had w/cm equal to 0.38. Mixture #4 (0.42-6.2-FA16-SF3.5-G) resulted in a slightly higher slump value than those mixtures with a w/cm equal to 0.38.

7.2.1.2 Supplementary Cementitious Materials

Fly ash is known to increase workability. Figure 7.1 shows Mixture #5 (0.44/6.5/FA30/II) with an increased w/cm of 0.44 and a 30% replacement percentage of cement with fly ash had significantly increased workability. In fact, Mixture #5 achieved the largest slump (8.0 in., 20.32 cm). This slump is higher than what is usually desirable in the field. Mixture #6 (0.44/6.5/FA30/SF5/II) is the same mixture but with a 5% replacement of cement with silica fume. Silica fume was expected to decrease workability and did so by 1.5 inches (3.81 cm). The 50% blast furnace slag mixture decreased workability significantly from the comparison mixtures #5 and #6 (5in. and 3.5in. respectively.

7.2.1.3 Chemical Admixtures

Mixtures #5 (0.44/6.5/FA30/II), Mixture #6 (0.44/6.5/FA30/SF5/II), Mixture #7 (0.44/6.5/BFS50/II), Mixture #8 (0.44-6.0-FA30-SRA-II), and Mixture #9 (0.44-6.0-FA30-RET-II) have a w/cm equal to 0.44 and did not require any HRWRA for workability. The chemical admixtures used in Mixtures # 8 and #9 did not result in increased workability.

7.2.1.4 Aggregate Type

Mixture #10 (0.42-6.0-II-L.W.A) and Mixture #11 (0.42-6.0-II-Norm.Wt.) with a w/cm equal to 0.42 required very little HRWRA. An advantage Mixture #10 (0.42-6.0-II-L.W.A) has over the other mixtures is the use of pre-soaked lightweight aggregate (L.W.A.). The additional water in the presoaked aggregate helped to increase slump (0.5in., 1.27 cm).

Each of the eleven mixtures attained adequate workability to mold all necessary test samples. Slump test results are shown in Figure 7.1.

7.2.2 Air Content

The use of an AEA was incorporated for all eleven mixtures. Current Class H and HT specifications require air content between 5% - 8%. The air content of the research mixtures varied throughout the research. The W.R. Grace air-entraining agent specifies a dosage of 1 fluid ounce per 100 pounds of cementitious materials. This dosage was measured correctly but resulted in random air contents. Previous research using the same dosage rate of AEA has repeatedly proven accurate air content results. The research team believes the error in air content to be caused by excessive cement replacement percentages with cementitious materials (Fly Ash) which caused unforeseen resulting air contents. Although trial batches were made to test the interaction between the various admixtures, the research team believes the interaction between chemical admixtures and high cementitious replacement percentages caused the design mixtures to have variable air contents.

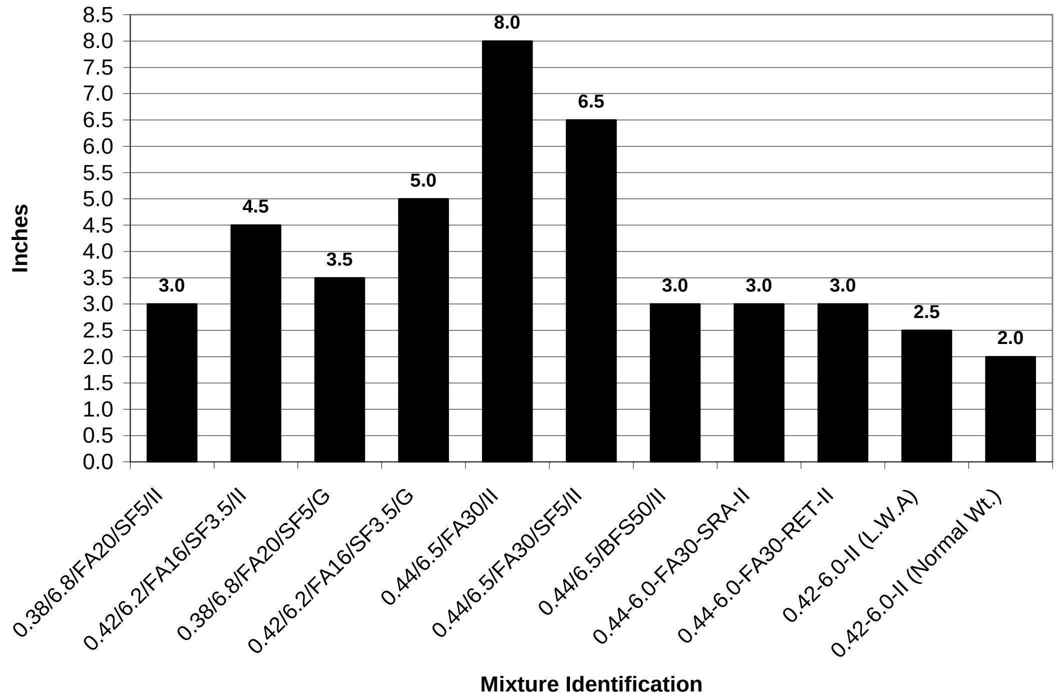

Figure 7.1 Slump Test Results (ASTM C 143, AASHTO T 119)

Mixture #3 (0.38-6.8-FA20-SF5-G) was batched first and the exact dosage was used for air content designed to be 6.5%. Mixture #3 (3.4%) is lower than the design of 6.5% by a margin of error equal to 48%. As a result, AEA dosages were re-evaluated for more accuracy. Mixture #1 and #2 were batched next. The AEA dosage was adjusted before batching Mixtures #1 and #2.

All of the mixtures using HRWRA required an amount different from the design to achieve adequate workability. The two mixtures having lower w/cm equal to 0.38 both required more than the design amount of HRWRA. As a result, the extended mixing time sometimes required to incorporate the HRWRA uniformly into the mixture essentially deflated the concrete, releasing the entrained air. This is typically the case with the mixtures having lower air contents than 6.5%.

Air contents also varied due to experimental replacement percentages of cement with supplementary cementitious materials and the use of chemical admixtures. These experimental mixtures sometimes had unexpected admixture interactions which were not anticipated during design. Air content values are provided in Figure 7.2.

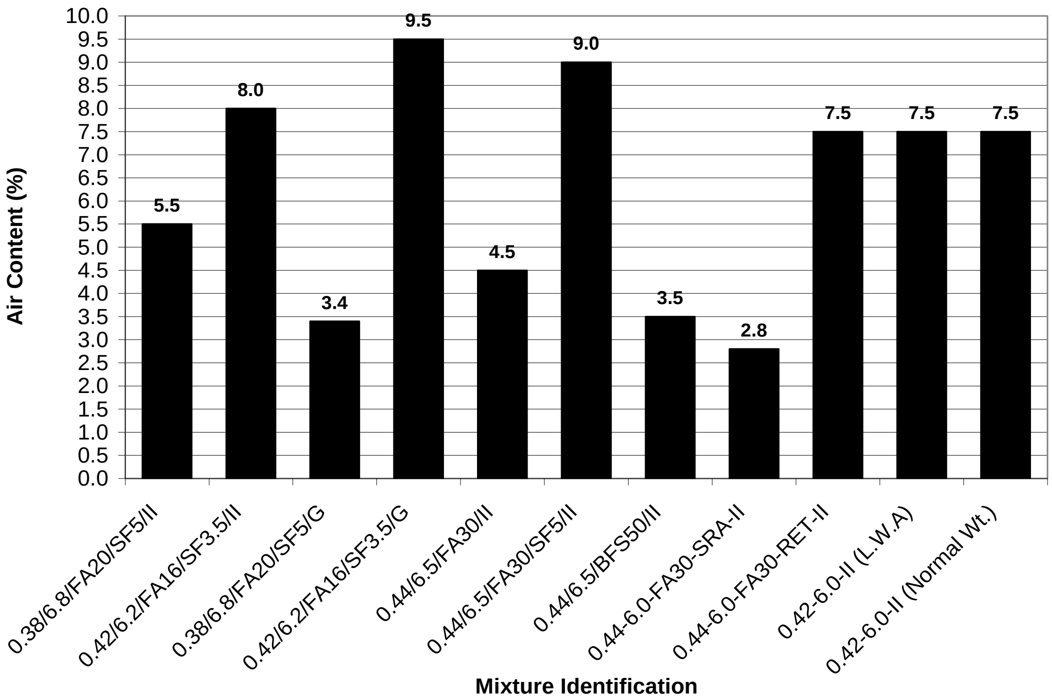

Figure 7.2 Air Content (ASTM C 231, AASHTO T 152)

7.2.3 Unit Weight

The unit weight of each mixture was determined at batching per ASTM C 138. The unit weight is the weight of a unit volume of concrete (Equation 2).

$$UnitWeight = \frac{WeightofConcrete(lbs.)}{ConcreteVolume(ft.^3)} \qquad \textbf{Equation 2}$$

The design unit weight was between 138 and 140.5pcf for all mixtures depending upon the amount of supplementary cementitious materials, w/cm, and resulting air content. The unit weight is affected by the air content and a direct relationship can be seen from the data. When the unit weight is greater than the design, the air content is lower than the design, and vice versa. The air content and the unit weight are inversely proportionate.

The unit weight of Mixture #1 (0.38-6.8-FA20-SF5-II) is 2 pounds heavier than the design while the air content is 1% less than design. Less air within the concrete translates to heavier materials filling the void spaces (i.e. sand, rock, cement paste).

Since the design unit weight for all mixtures was between 138 and 140.5pcf, and 6.5% air content, the same trend can be seen in all mixtures from the data above. Any mixture having air content higher than 6.5% has a unit weight lower than the design of 140.5pcf and vice versa. Again, the air content and the unit weight are inversely proportionate. The various air contents resulted in unit weights both above and below the 6.5% design. A comparison between air content and unit weight is shown for each mixture in Figure 7.3.

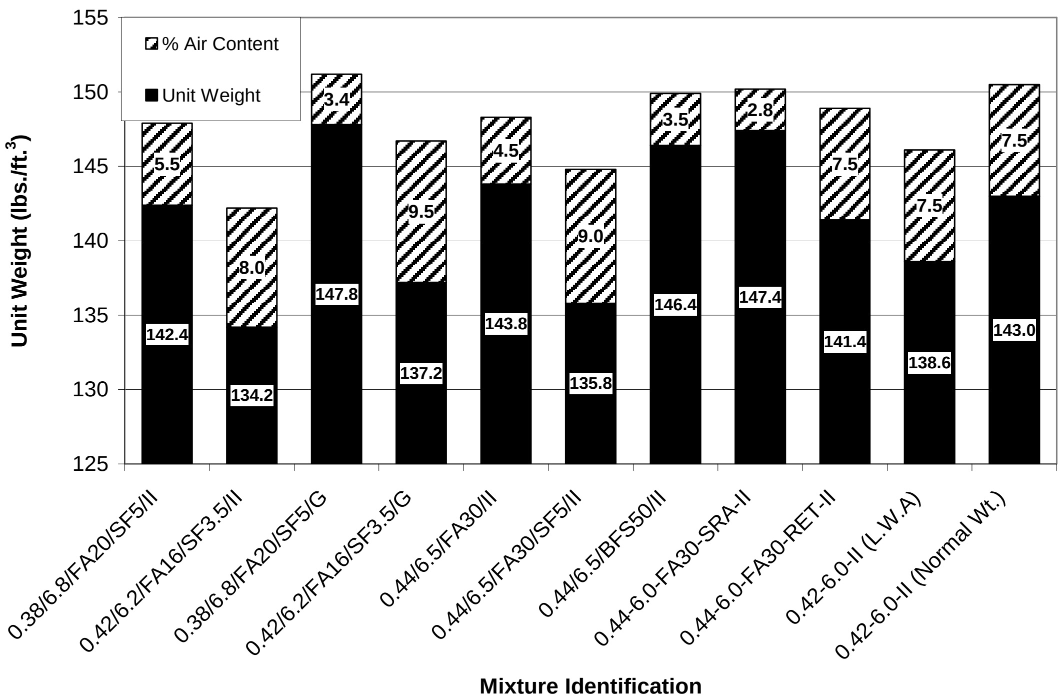

Figure 7.3 Unit Weight (ASTM C 138, AASHTO T 121) vs. Air Content (ASTM C 231, AASHTO T 152)

7.2.4 Concrete Temperature

The ideal temperature to place concrete is between 50 and 60 degrees Fahrenheit (10 to 16 degrees Celsius), but should not exceed 85 degrees Fahrenheit (29 degrees Celsius) (Mindess et al, 2003). Excessive temperatures in concrete cause an increase in the evaporation of water from the concrete. This undesirable increased rate of evaporation is the cause of plastic shrinkage and results in internal, crack-causing stresses. The concrete temperature for the research mixtures ranged from 58 to 72 degrees Fahrenheit (14 to 22 degrees Celsius). None of the concrete temperatures exceeded the recommended maximum temperature. Concrete temperatures are shown in Figure 7.4. Concrete temperatures were assumed to be acceptable for design performance.

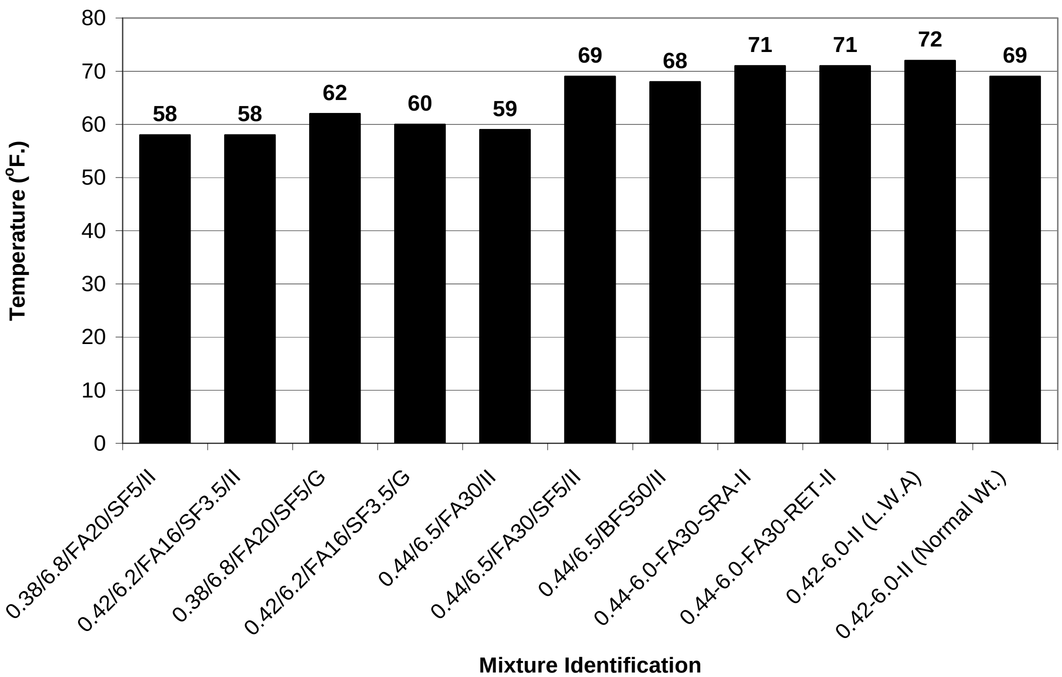

Figure 7.4 Concrete Temperature, (ASTM C 1064, AASHTO T 309)

7.3 Hardened Concrete Tests

Hardened concrete tests performed for this study included compressive strength, restrained shrinkage, permeability, and freeze/thaw durability.

7.3.1 Compressive Strength

Compressive strength is an important design aspect of concrete. More importantly, field performance of the designed compressive strength is imperative. Compressive strength was tested for each mixture at 1, 3, 7, 28, and 56-days of age. Three cylinders were tested for each mixture on the respective day of age. The compressive strength was found by dividing the compressive load at failure by the surface area of the concrete cylinder tested (Equation 3). The cylinders were of 4in x 8in (10.16cm x 20.32cm, radius x diameter) dimensions. Figure 7.5 depicts a cylinder being tested.

Compressive Strength: $$f'c = \frac{Load\,(lbs.)}{Area\,(in.^2)}$$ **Equation 3**

Figure 7.5 Photograph of Compressive Strength Failure (ASTM C 39, AASHTO T 22)

Mixtures were designed for laboratory research and ideal conditions. According to current CDOT Class H and HT specifications, the laboratory trial mixture for Class H or HT concrete must produce an average 56-day compressive strength at least 115 percent of the required 56-day field compressive strength (Equation 4).

$$f'c + 1.15*f'c$$ **Equation 4**

Current CDOT Class H and HT specifications require a 56-day compressive strength of 4500psi. As a result, the mixtures designed for this research had a design compressive strength of 5175psi. Compressive strengths for all design mixtures are shown in Table 7.2.

Table 7.2 Compressive Strength (ASTM C 39, AASHTO T 22)

Mixture	Mixture	AGE				
Number	Identification	1-day	3-day	7-day	28-day	56-day
		lbs./in.^{2}				
1	0.38/6.8/FA20/SF5/II	2135	3880	4632	5778	6479
2	0.42/6.2/FA16/SF3.5/II	1216	2644	3182	4161	4643
3	0.38/6.8/FA20/SF5/G	1369	3879	5232	7621	8712
4	0.42/6.2/FA16/SF3.5/G	601	1437	2266	3472	3931
5	0.44/6.5/FA30/II	974	2575	3422	4764	5467
6	0.44/6.5/FA30/SF5/II	876	1886	2653	3816	4298
7	0.44/6.5/BFS50/II	881	3382	5346	6662	6976
8	0.44-6.0-FA30-SRA-II	1392	2932	3496	4817	5685
9	0.44-6.0-FA30-RET-II	1404	3281	3637	4806	5572
10	0.42-6.0-II (L.W.A)	2844	4347	4754	5807	6273
11	0.42-6.0-II (Normal Wt.)	2935	4746	5003	5678	5869

7.3.1.1 Mixtures Having Inadequate 56-Day Strength

Current CDOT Class H and HT specifications require a compressive strength of 4500 psi at 56-days of age. In practice these strengths are sometimes achieved as early as 7 days of age. Other state DOT's require only 3500psi at 56-days of age and feel this is adequate strength for bridge decks. Figure 7.6 shows the 56-day compressive strength results for all mixtures compared to the current Class H and HT requirement.

Increased air content results in decreased compressive strength. The compressive strength of concrete is reduced by approximately 5% for each 1% increase in air content (Mindess, Young, and Darwin, 2003). By having an increase of 3.0 and 2.5% air, the compressive strength of the mixtures would decrease by 15 and 12.5%, respectively. Mixture #4 had a 56-day compressive strength of 3931 psi, of which 15% is 590psi, totaling 4521psi. Mixture #6 had a 56-day compressive strength of 4298psi, of which

12.5% is 452psi, totaling 4973psi. This process is referred to as normalizing data. The normalization of compressive strength for air content shows a sufficient strength for these two design mixtures when air content is accurately incorporated into the mixture. Compressive strengths normalized for air content are discussed further section 7.4.2.2.

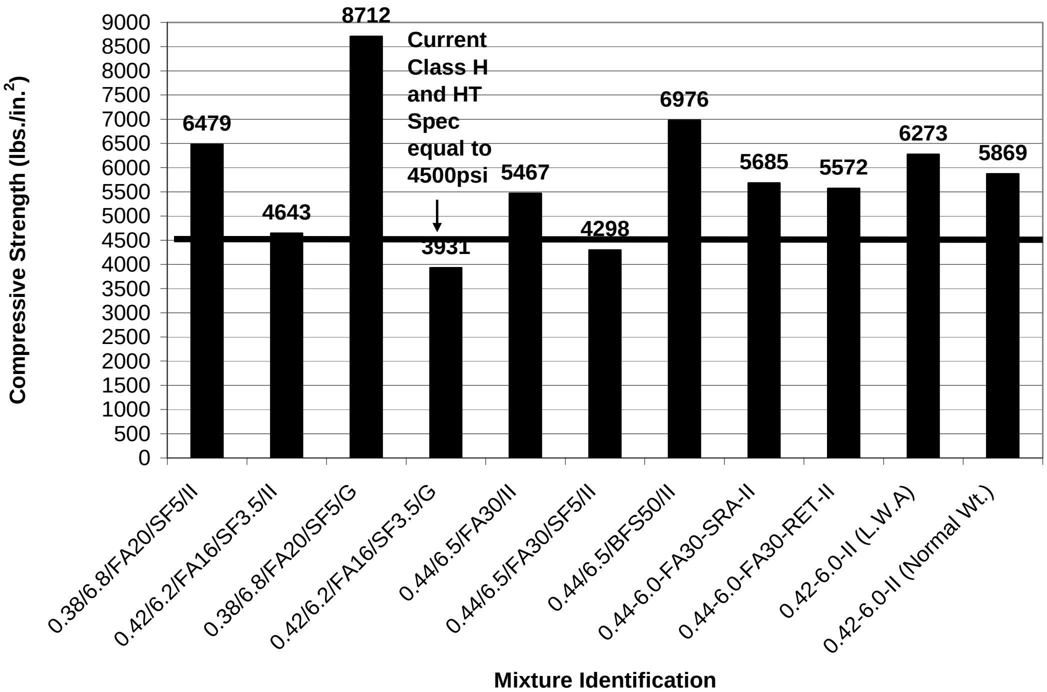

Figure 7.6 56-Day Compressive Strength (ASTM C 39, AASHTO T 22)

Two of the design mixtures did not satisfy the 56-day compressive strength requirement. Both had air contents in excess of the design by 3.0% and 2.5%, or 9.5% and 9.0% respectively.

7.3.1.2 Normalization of Compressive Strength

The air content for the mixtures varied from the design of 6.5%. Various air contents resulted from the use of chemical admixtures, supplementary cementitious materials contents, and the resulting mixing times necessary to achieve adequate workability of the mixture. As mentioned previously, the compressive strength and the air content are inversely proportionate; as air content increases compressive strength decreases. In fact, the compressive strength of concrete is decreased 5% for each 1% increase in air content

(Mindess, Young, and Darwin, 2003). Normalized, 56-day compressive strength results accounting for either a higher or lower air content from the design are shown in Table 7.3 and Figure 7.7.

Table 7.3 Normalized Compressive Strength

Mixture	Mixture	Air	Deign Air	Age				
Number	Identification	Content	Content	1-day	3-day	7-day	28-day	56-day
		(%)	(%)	lbs./in.^{2}				
1	0.38/6.8/FA20/SF5/II	5.5	6.5	2028	3686	4401	5489	6155
2	0.42/6.2/FA16/SF3.5/II	8.0	6.5	1307	2842	3420	4473	4991
3	0.38/6.8/FA20/SF5/G	3.4	6.5	1157	3278	4421	6440	7362
4	0.42/6.2/FA16/SF3.5/G	9.5	6.5	691	1653	2606	3993	4521
5	0.44/6.5/FA30/II	4.5	6.5	876	2318	3080	4288	4920
6	0.44/6.5/FA30/SF5/II	9.0	6.5	986	2121	2985	4293	4835
7	0.44/6.5/BFS50/II	3.5	6.5	748	2874	4544	5663	5930
8	0.44-6.0-FA30-SRA-II	2.8	6.5	1135	2389	2849	3926	4633
9	0.44-6.0-FA30-RET-II	7.5	6.5	1474	3445	3819	5047	5851
10	0.42-6.0-II (L.W.A)	7.5	6.5	2986	4564	4992	6097	6587
11	0.42-6.0-II (Normal Wt.)	7.5	6.5	3082	4983	5254	5962	6162

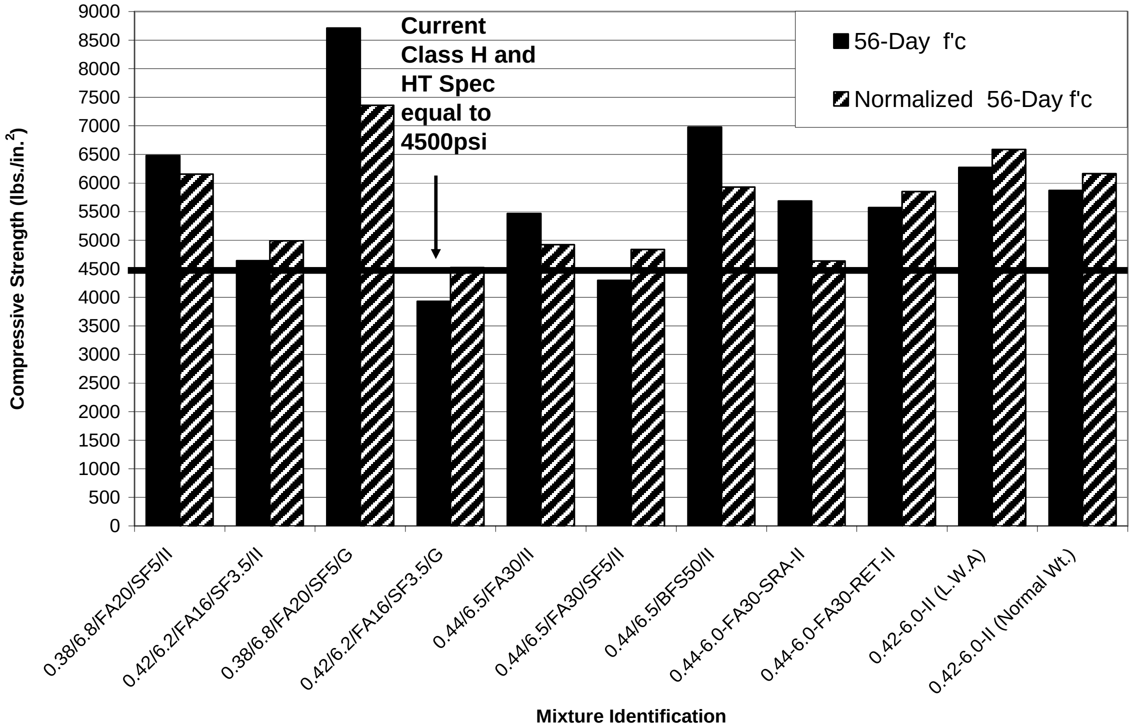

Figure 7.7 56-Day Compressive Strength vs. 56-Day Compressive Strength (Normalized for Air Content), (ASTM C 39, AASHTO T 22)

When normalized for air content, all eleven mixtures achieved the current CDOT Class H and HT field specification requiring 4500psi at 56-days of age.

7.3.1.3 Comparison of Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #2 (0.42-6.2-FA16-SF3.5-II), Batch One and Two

A second batch of mixtures 1 and 2 were made to cast new restrained shrinkage rings because the data logger stopped recording strain after an insufficient period of time. The restrained shrinkage specimens fabricated during the second batching of mixtures one and two (0.38-6.8-FA20-SF5-II and 0.42-6.2-FA16-SF3.5-II, respectively) were used to conduct restrained ring shrinkage tests. The other test specimens (freeze/thaw, permeability, compressive strength) were fabricated during the first batch of mixtures # 1 and #2. Since specimens for the same mixture were fabricated at two different batch times, a comparison of compressive strength was performed for each mixture, batch one and two. Early-age compressive strength results are shown for Mixture #1 and Mixture #3 in Figures 7.8 and 7.9, respectively.

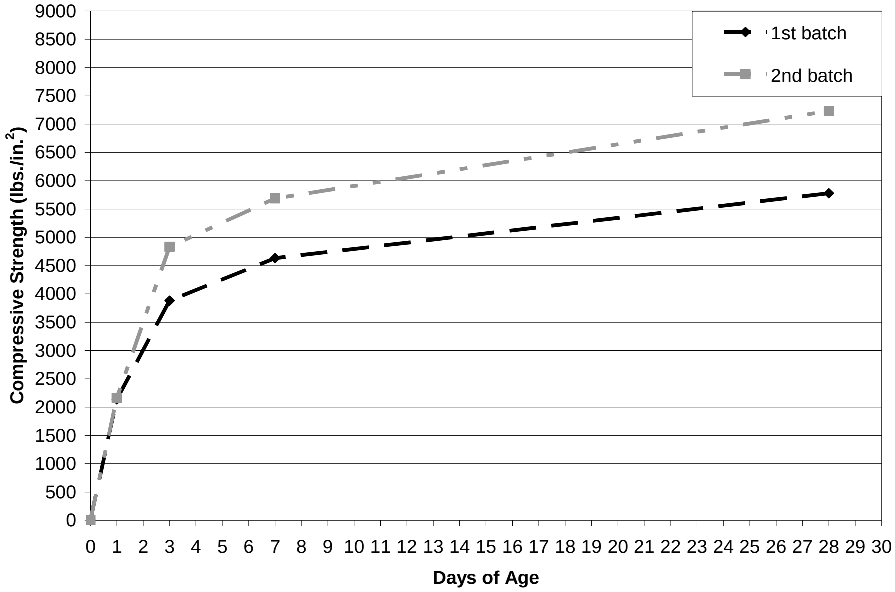

Figure 7.8 28-Day Compressive Strength, CDOT Control Mixture #1 (0.38-6.8-FA20-SF5-II), Batch One vs. Batch Two (ASTM C 39, AASHTO T 22)

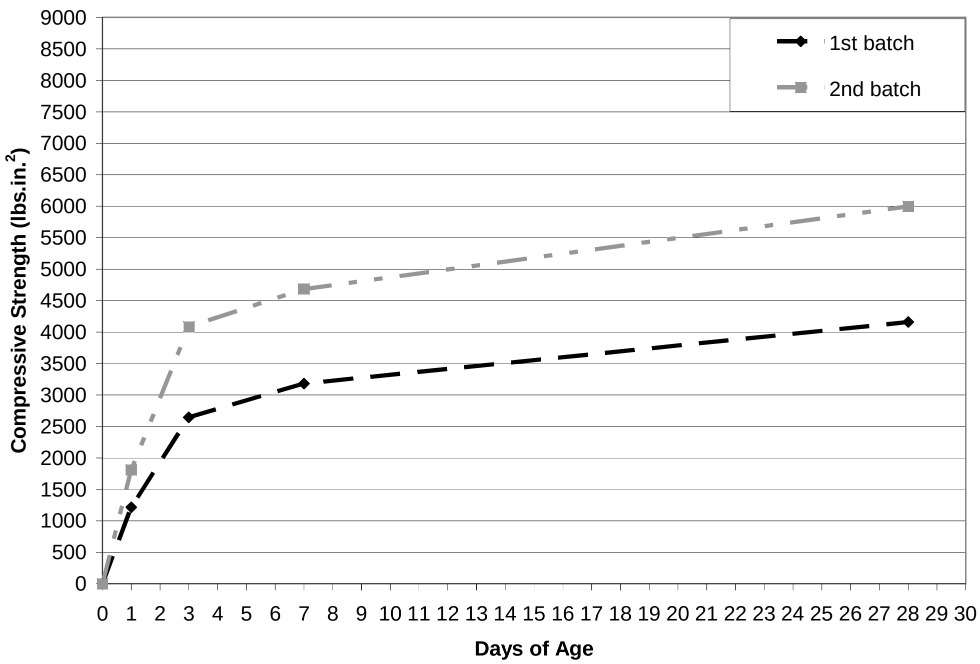

Figure 7.9 28-Day Compressive Strength, CDOT Control Mixture #2 (0.42-6.2-FA16-SF3.5-II), Batch One vs. Batch Two (ASTM C 39, AASHTO T 22)

When making the second batch of mixture 1, the coarse-aggregate supply was nearly diminished and contained noticeably more fines in its composite. Mixture 1 (0.38-6.8-FA20-SF5-II), batch two, demonstrated an increase of less than 1% compressive strength at 1-day of age over batch 1 (2165 vs. 2161psi). At 3-days of age the compressive strength of batch two had increased 20% over batch 1 (4831 vs. 3880 psi) and 18% at 7 days of age (5680 vs. 4632 psi). This trend continued as the compressive strength at 28-days of age was 20% higher for batch two than batch one (7234 vs. 5778 psi). It should be noted that at 7days of age, mixture one, batch two achieved within 2% of the compressive strength as batch one achieved at 28-days of age.

Mixture #2 (0.42-6.2-FA16-SF3.5-II), batch two was batched immediately following the re-batching of Mixture #1 (0.38-6.8-FA20-SF5-II), batch 2. Again, the coarse aggregate contained noticeably more fines in its composite. It contained an even slightly higher amount of fines than the re-batch for mixture one (0.38-6.8-FA20-SF5-II). The remaining coarse-aggregate supply was churned to ensure consistency and uniformity of the last of the rock. There was more than enough coarse-aggregate to satisfy batch weights so the concrete was made and test specimens fabricated. The results show an increased compressive strength between batches one and two of both mixtures.

Mixture two (0.42-6.2-FA16-SF3.5-II), batch two, achieved 33% increased compressive strength at 1-day of age than batch one (1812 psi vs. 1216 psi) and 35% by 3-days of age (4086psi vs. 2644 psi). By 7 and 28-days of age the second batch had achieved 32 and 31% more compressive strength than batch one (4684 vs. 3182psi and 5998 vs. 4161psi. respectively). It should be noted that Mixture #2, batch two achieved the same compressive strength at 3-days of age as batch one at 28-days of age.

The increased amount of fines in the coarse-aggregate is believed to be the cause for the increased compressive strength observed between batches one and two. The fines act as a source of strength in concrete and the increased amount of fines would have replaced a portion of the larger aggregate. This results in a more dense concrete structure with an increased compressive strength.

The second batch of mixtures 1 and 2 were made to cast new restrained shrinkage rings because the data logger stopped recording strain after an insufficient period of time.

The test specimens for permeability, freeze/thaw durability and strength were originally made during batch one of mixtures 1 and 2. The batch two cylinder specimens were made to show a similarity in compressive strength so that data from two different batch times (of the same mixture) would be accepted for the purposes of this study. However, the increased fines in the coarse aggregate created strengths beyond that of batch one of mixtures 1 and 2, creating errors in the comparison. It should be noted that both batches one and two for each mixture remade were identical in batch quantities.

7.3.1.4 Early-Age Compressive Strength

Compressive strength varied from mixture to mixture at respective days of testing. Supplementary cementitious materials, the type of cement, and the use of chemical admixtures affect the rate of strength gain at both early and late stages of concrete age. A comparison of the early-age compressive strength and rate of strength gain is of interest when researching shrinkage strain. An increased rate of strength development will result in increased concrete stresses, often leading to cracking (Xi et al, 2001). Compressive strength results and the development of strength will also be discussed in the section analyzing shrinkage strain data for the purposes of this study. A comparison of early-age strength gain for all mixtures through 7 days of age is shown in Figure 7.10.

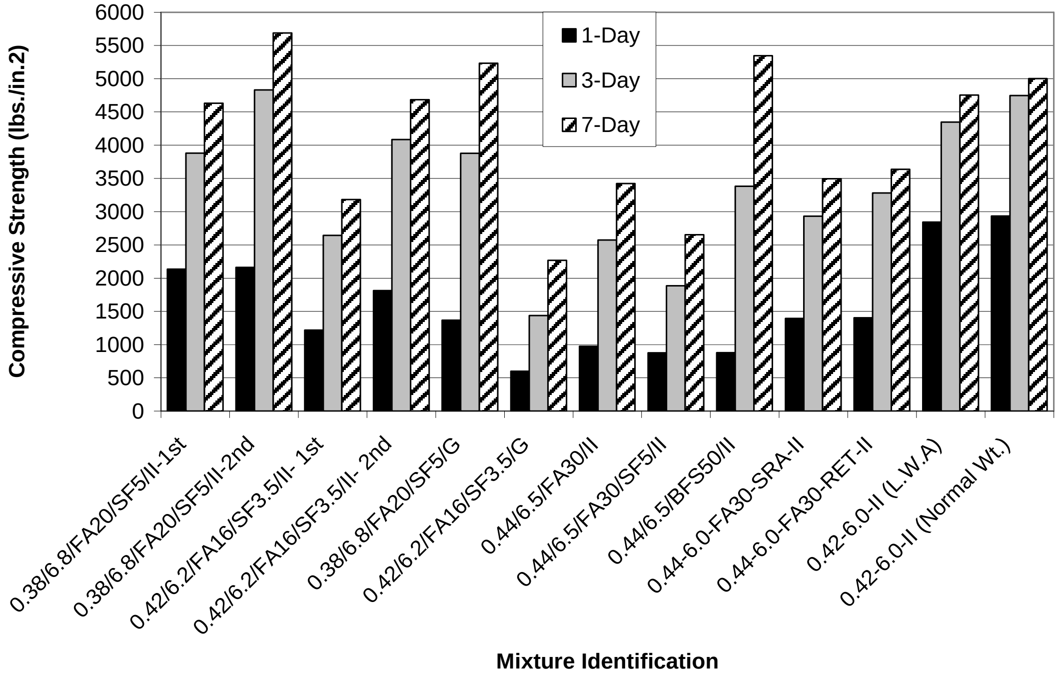

Figure 7.10 Early-Age Compressive Strength (ASTM C 39, AASHTO T 22)

7.3.1.4.1 Cement Type

Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #3 (0.38-6.8-FA20-SF5-G) are identical in batch quantities but they are made using different cement: Type II cement and Class G oil well, coarse-ground cement, respectively. Coarse-ground cement is expected to decrease the rate of strength gain. Also accompanied by a lower heat of hydration, the concrete is expected to develop lower thermal stresses at early ages and therefore, be less susceptible to cracking. It is expected that the Type G cement will gain early-age strength at a rate slower than the Type II mixture. Data illustrating the early-age strength gain of the CDOT Class H and HT mixtures made using the typical Type II cement versus the coarse ground cement are plotted in Figures 7.11 and 7.12, respectively.

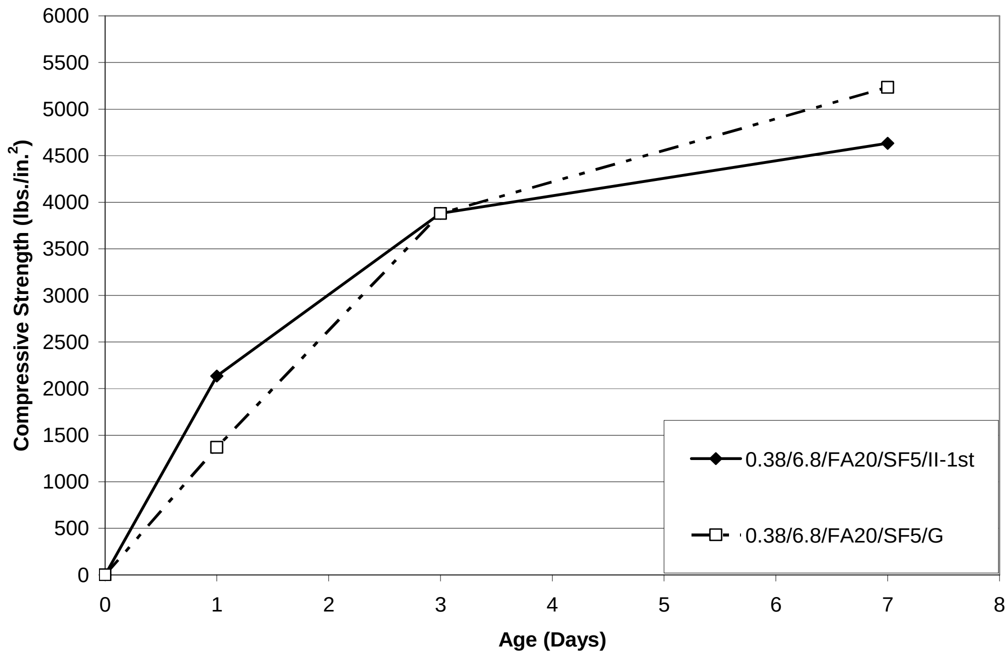

Figure 7.11 Early-Age Compressive Strength, CDOT Control Mixture #1 (0.38-6.8-FA20-SF5-II) (Type II Cement) and Mixture #3 (0.38-6.8-FA20-SF5-G) (Type G, Coarse-Ground Cement) (ASTM C 39, AASHTO T 22)

As expected at 1-day of age, Mixture #1 (0.38-6.8-FA20-SF5-II) made with Type II cement gained 36% more compressive strength than Mixture #3 (0.38-6.8-FA20-SF5-G) made using coarse-ground cement, 2135 vs. 1369psi. At the same age, Mixture #4 (0.42-6.2-FA16-SF3.5-G), developed only 51% of the compressive strength achieved by Mixture #2 (0.42-6.2-FA16-SF3.5-II) made using Type II cement, 601 vs. 1216psi. This data shows that coarse-ground cement gains strength at a slower rate than Type II cement at 1-day of age. Mixtures #2 and #4 have a w/cm equal to 0.42 (higher than Mixtures #1 and #3- 0.38) and as expected, are gaining strength at a slower rate than Mixtures #1 and #3.

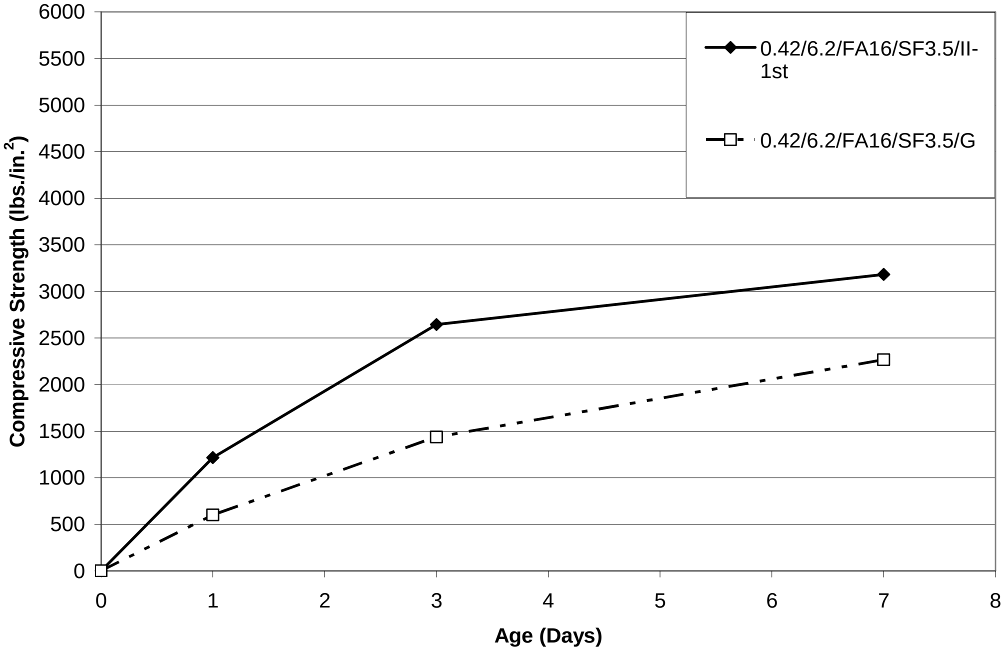

Figure 7.12 Early-Age Compressive Strength, CDOT Control Mixture #2 (0.42-6.2-FA16-SF3.5-II) (Type II Cement) and Mixture #4 (0.42-6.2-FA16-SF3.5-G) (Type G, Coarse-Ground Cement) (ASTM C 39, AASHTO T 22)

However, by 3-days of age, Mixture #3 (0.38-6.8-FA20-SF5-G) recovered to gain as much strength as its Type II counterpart Mixture #1 (0.38-6.8-FA20-SF5-II), and surpassed the Type II mixture, 3880 vs. 3879psi. The magnitude of the two mixtures is the same at 3 days of age but their respective percentages of ultimate strength acquired are significantly different, 60% vs. 45% respectively.

At three days of age Mixture #3 (0.38-6.8-FA20-SF5-G) achieved 51% of its 28-day strength while Mixture #1 (0.38-6.8-FA20-SF5-II) achieved 33% of its respective 28-day compressive strength. However, the Type G cement concrete mixture achieved an almost identical magnitude of compressive strength at 3-days of age compared to its Type II counterpart (3880 psi vs. 3879.2 psi). The Type G cement proves to better regulate the rate of strength gain at 3-days of age and younger. As expected, the higher w/cm mixtures continue gaining strength at a slightly slower rate. This slower rate of strength gain will reduce thermal stresses and cracking potential.

By 3 days of age Mixture #2 (0.42-6.2-FA16-SF3.5-II) achieved 46% more compressive strength than Mixture #4 (0.42-6.2-FA16-SF3.5-G), 2644 vs. 1437psi. At 7 days of age, the increased compressive strength of Mixture #2 had been reduced to 29%, 3182 vs. 2266psi.

At 7 days of age and younger and with an increased w/cm the Type G cement hydrates more slowly. At 7-days of age the coarse-ground cement began to gain strength at a similar rate to the Type II mixtures. The four mixtures (0.38-6.8-FA20-SF5-II, 0.42-6.2-FA16-SF3.5-II, 0.38-6.8-FA20-SF5-G, and 0.42-6.8-FA16-SF3.5-G) have achieved 80%, 77, 69%, and 65% respectively, of their 28-day compressive strength. The coarse ground cement mixtures continue achieving a slightly slower rate of strength gain. Strength development trends continue through 7-days of age. At 7-days of age Mixture #4 (0.42-6.2-FA16-SF3.5-G) achieved the lowest compressive strength (2266psi.). This is due to the increased w/cm (0.42) in conjunction with a low percentage replacement of cementitious materials while using Type G cement.

7.3.1.4.2 Supplementary Cementitious Materials

At 7-days of age the increased air content in Mixture #6 (0.44-6.5-FA30-SF5-II) continued to reduce its strength gain less than its counterpart (Mixture #5 (0.44-6.5-FA30-II)). The silica fume replacement typically increases the strength of concrete however; the increased air content has super ceded the 5% replacement of cement with silica fume and reduced the compressive strength by 22% (3422 vs. 2653psi.).

Mixtures # 5, #6, and #7 all have a relatively low 1-day compressive strength (<1000 lbs/.in.^{2}). See Figure 7.13. This is due in part to the increased w/cm equal to 0.44 for all three mixtures. This increased water will reduce the compressive strength throughout the life of the concrete and was incorporated by the research team to reduce the early and long-term strength of the concrete. As shown in Mixtures #1 and #3, the current CDOT Class H and HT mixtures produce 28-day compressive strengths well above the required. Mixtures #5 (0.44-6.5-FA30-II)and #6 (0.44-6.5-FA30-SF5-II) are similar except Mixture #6 introduces a 5% replacement of cement with silica fume in addition to the original 30% fly ash replacement. It is the increased air content of 9% vs.

4.5% that has reduced the 1-day compressive strength of Mixture #6 by 10% of Mixture #5 (974psi vs. 876psi).

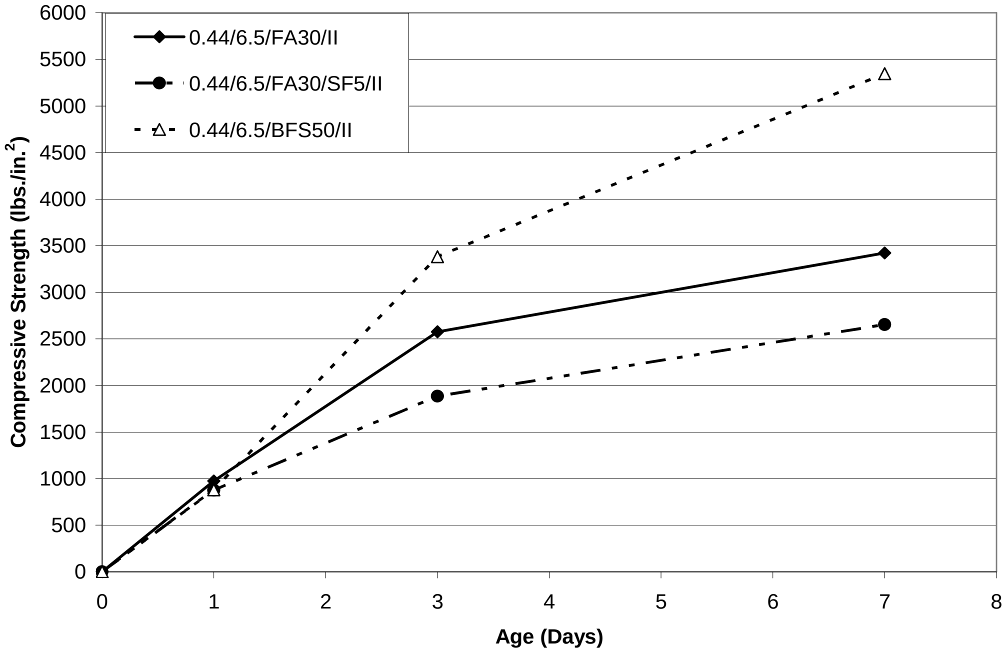

Figure 7.13 Early-Age Compressive Strength, Mixture #5 (0.44-6.5-FA30-II), Mixture #6 (0.44-6.5-FA30-SF5-II), and Mixture #7 (0.44-6.5-BFS50-II) (ASTM C 39, AASHTO T 22)

At 1-day of age Mixture #7 (0.44-6.5-BFS50-II) gained strength within 1% of Mixture #6 (0.44-6.5-FA30-SF5-II). This is the only design mixture utilizing blast furnace slag in this research.

At 3-days of age Mixture #6 (0.44-6.5-FA30-SF5-II) gained strength at a slower rate than its counterpart (Mixture #5 (0.44-6.5-FA30-II)) containing 30% fly ash and no silica fume. The air content reduced the 3-day strength by 27% (1886 vs. 2575psi). At 3-days of age, Mixture #7 (0.44-6.5-BFS50-II) began to develop strength more rapidly than Mixtures #5 and #6. In fact, the blast furnace slag mixture increased the 3-day compressive strength by 24% and 44% beyond that of Mixtures #5 and #6 (3382 psi. vs. 2575psi. or 1886psi.).

The compressive strength between mixtures #6 and #7 is skewed. Air contents for these mixtures varied from 9% for Mixture #6 (0.44-6.5-FA30-SF5-II) and 3.5% for

Mixture #7 (0.44-6.5-BFS50-II). As a result, the compressive strength for Mixture #6 is lower than designed and the compressive strength is higher than designed for Mixture #7. The compressive strength results for these mixtures would be closer to one another if the 6.5% air content the mixtures were designed with had been achieved. The air content is believed to have varied slightly due to the fly ash and the blast furnace slag percentage replacements.

At 7-days of age Mixture #7 (0.44-6.5-BFS50-II) continued its accelerated strength gain and surpassed Mixtures #5 and #6 by 36% and 50% (5346 vs. 3422 psi and 2653 psi).

7.3.1.4.3 Chemical Admixtures

At 1 day of age Mixture #8 (0.44-6.0-FA30-SRA-II) (Shrinkage Reducing Admixture) and Mixture #9 (0.44-6.0-FA30-RET-II) (Set Retarding Admixture) have respective compressive strengths of 1392 and 1402psi. At one day of age the set retarder begins to allow hydration to occur and the rate of strength gain began to increase for Mixture #9 (0.44-6.0-FA30-RET-II. At 3 days of age, the shrinkage reducing mixture achieved only 89% of the set retarder mixture, 2932 vs. 3281psi.

From 3 to 7 days of age the rate of strength gain between the two mixtures is comparable but different in magnitude. The rate of strength for the shrinkage reducing mixture began to increase at 3 days of age.

At 7 days of age, Mixture #9 (0.44-6.0-FA30-RET-II) only achieved a 4% increased compressive strength over Mixture #8 (0.44-6.0-FA30-SRA-II). Respective 7 day compressive strengths were 3637 and 3496psi.

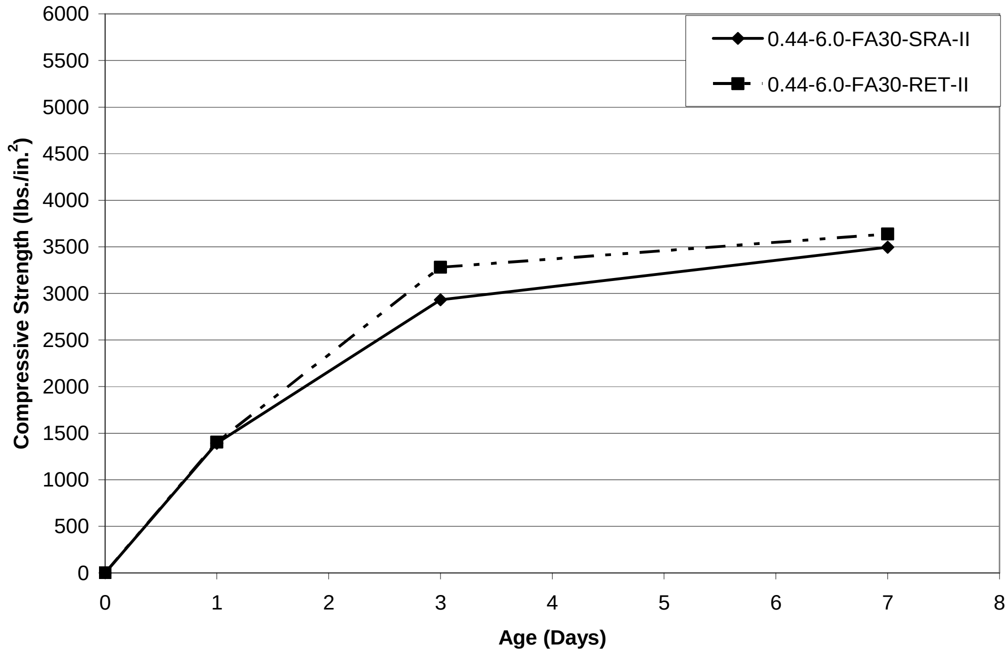

Figure 7.14 Early-Age Compressive Strength, Mixture #8 (0.44-6.0-FA30-SRA-II) (Shrinkage Reducing Admixture) and Mixture #9 (0.44-6.0-FA30-RET-II) (Set Retarding Admixture) (ASTM C 39, AASHTO T 22)

7.3.1.4.4 Aggregate Type

Mixture #10 ((0.42-6.0-II) made with Lightweight Aggregate (LWA) and Mixture #11 (0.42-6.0-II) made with Normal Weight Aggregate (NWA) were compared to investigate the effect of internal curing by incorporating the use of pre-soaked, lightweight aggregate (sand).

It should be noted that concrete made using lightweight aggregate is not lightweight concrete. The unit weight of the mixture made using lightweight aggregate falls within the range of normal weight concrete (138.6 lbs./ft.^{3}).

At the time of batching, the pre-soaked lightweight aggregate had a moisture content of 18%. The increased moisture was expected to effectively help cure the concrete internally. This internal curing was intended to help reduce restrained shrinkage strain in the concrete as it ages. The LWA sand releases moisture back into the mixture rather than absorbing mixture water during hydration. The LWA is also expected to help the hydration process at ages beyond 7 days (Cusson and Hooegeveen, 2006).

As shown in Figure 7.15, at 1 day of age both the normal weight aggregate mixture and the lightweight aggregate (sand) mixtures gain strength at a similar rate. Mixture #10 ((0.42-6.0-II) made with LWA. and Mixture #11 (0.42-6.0-II) made with NWA. achieved compressive strengths within 3% of one another; 2844psi vs. 2935psi, respectively.

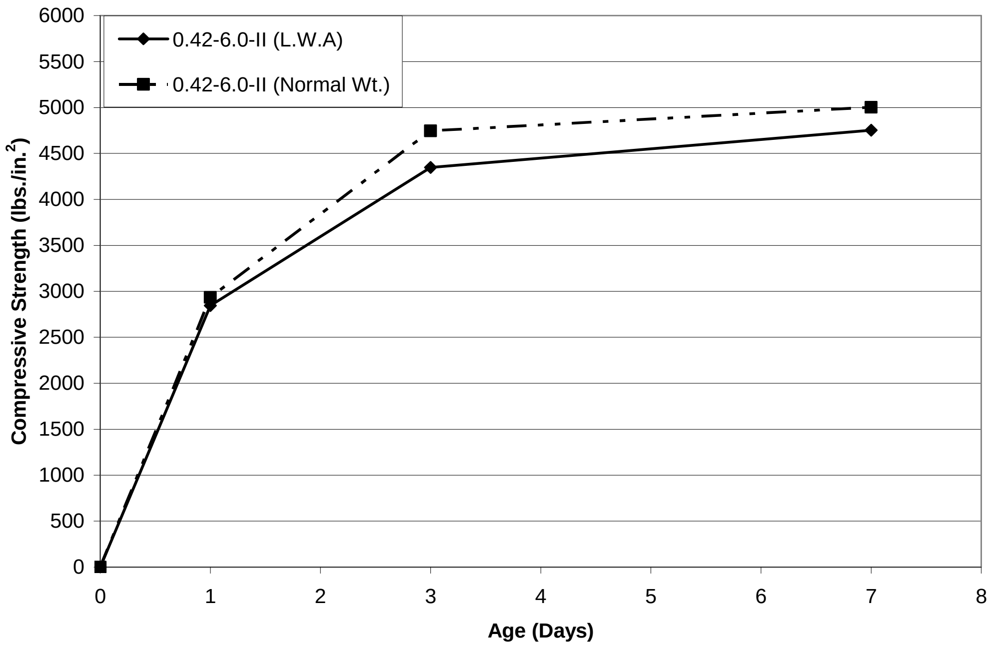

Figure 7.15 Early-Age Compressive Strength, Mixture #10 (0.42-6.0-II-Lightweight Aggregate) and Mixture #11 (0.42-6.0-II-Normal Weight Aggregate), (ASTM C 39, AASHTO T 22)

Between 3 and 7 days of age, the normal weight mixture began to gain strength at a slightly faster rate than, but still similar to, the lightweight aggregate mixture. By 7 days of age, the internally cured Mixture #10 (0.42-6.0-II-LWA.) had achieved 4754psi when Mixture #11 (0.42-6.0-II-NWA.) reached 5003psi; a 5% difference.

7.3.1.5 Ultimate Strength (28-Day and 56-Day)

The rate of strength gain varies for all mixtures from 1 through 56-days of age. In many of the comparisons, the rate of strength gain for one mixture was increased over another and this changed as the concrete aged. The following sections discuss the compressive strength and the rate of strength gain at 28 and 56-days of age.

7.3.1.5.1 Cement Type

At 28 and 56-days of age, respectively, the Type G cement Mixture #3 (0.38-6.8-FA20-SF5-G) achieved 32 and 34% more compressive strength than the Type II cement Mixture #1 (0.38-6.8-FA20-SF5-II). See Figure 7.16. This shows that at lower w/cm equal to 0.38 the Type G cement can in fact control the rate of early-age strength gain, while it does not jeopardize the ultimate strength of the concrete. The slower rate of strength gain should also result in a lower heat of hydration and, in turn, lower thermal stresses, which can help to decrease early-age cracking in concrete. However at higher w/cm equal to 0.42, both the early age strength gain and ultimate strength of the concrete is reduced by Type G, coarse-ground cement. See Figure 7.17. At this w/cm, the Type G mixture achieved 17% (4161 vs. 3472 psi.) and 15% (4643 vs. 3931 psi.) less compressive strength at 28 and 56-days than its Type II counterpart, respectively.

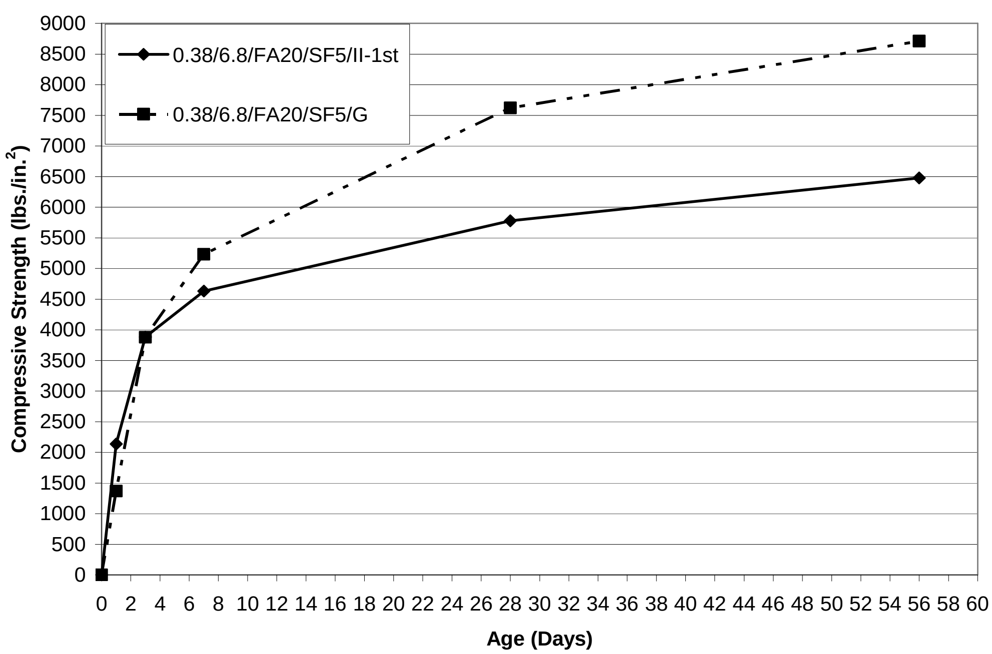

Figure 7.16 Compressive Strength, CDOT Control Mixture #1 (0.38-6.8-FA20-SF5-II) (Type II Cement) and Mixture #3 (0.38-6.8-FA20-SF5-G) (Type G, Coarse-Ground Cement), (ASTM C 39, AASHTO T 22)

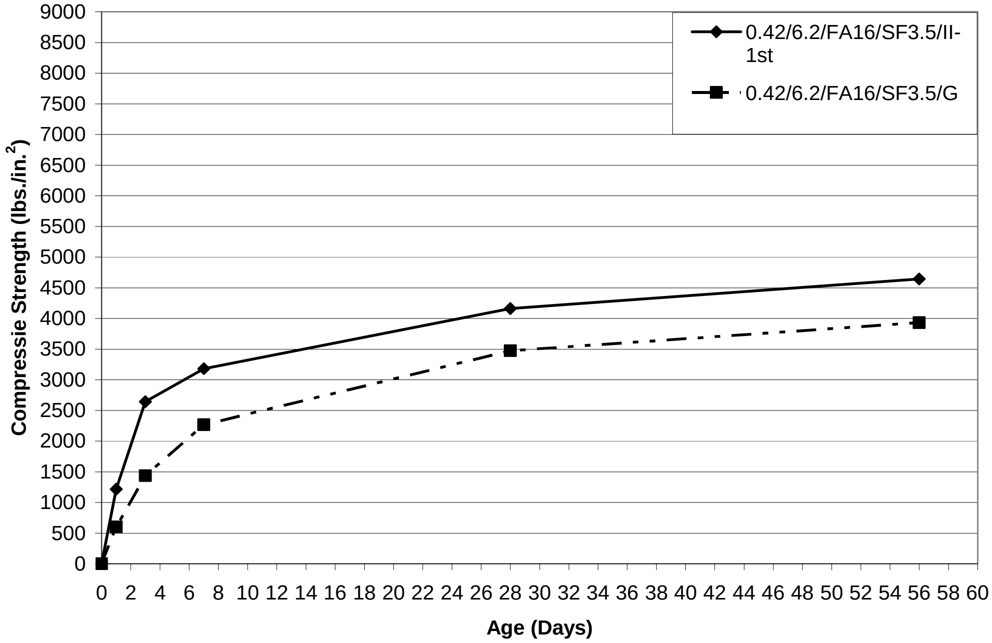

Figure 7.17 Compressive Strength, CDOT Control Mixture #2 (0.42-6.2-FA16-SF3.5-II) (Type II Cement) and Mixture #4 (0.42-6.2-FA16-SF3.5-G) (Type G, Coarse-Ground Cement), (ASTM C 39, AASHTO T 22)

As mentioned above this reduction in rate of strength gain will result in lower thermal stresses and is expected to help reduce restrained-shrinkage cracking. In addition, the reduction in rate of ultimate strength gain should help reduce restrained-shrinkage cracking beyond early-ages of concrete.

7.3.1.5.2 Supplementary Cementitious Materials

At 28-days of age the silica fume in Mixture #6 (0.44-6.5-FA30-SF5-II) would be expected to increase the rate of strength gain over its counterpart (Mixture #5 (0.44-6.5-FA30-II)). At 28-days of age the silica fume mixture resulted in lower compressive strengths than its counterpart by 20% (3816 vs. 4764 psi.). This trend accompanies the silica fume mixtures due to the high air content of the mixture previously mentioned. As a result, the accelerated strength gain typically associated with silica fume has been removed. Shown in Figure 7.18, the strength gain through 56-days of age for Mixture #5 (0.44-6.5-FA30-II) and #6 (0.44-6.5-FA30-SF5-II) are consistent. At 56-days of age, Mixture #6 (0.44-6.5-FA30-SF5-II) achieved only 79% of the ultimate strength (4298 vs.

5467psi) reached by the mixture made with fly ash and cement alone Mixture #5 (0.44-6.5-FA30-II).

At 28-days of age the 50% blast furnace slag Mixture #7 (0.44-6.5-BFS50-II) continued to surpass Mixtures #5 and #6 by 28% (6662 psi. vs. 4764 psi.) and 43% (6662 psi. vs. 3816 psi.), respectively.

At 56-days of age, the 50% blast furnace slag achieved an ultimate compressive strength higher than Mixture #5 (0.44-6.5-FA30-II) and #6 (0.44-6.5-FA30-SF5-II) by 22% (6976 vs. 5467psi) and 38% (6976 vs. 4298psi), respectively.

Again, it is clear from the data that blast furnace slag greatly increases the rate of strength gain beyond 7-days of age and this may contribute to increased restrained-shrinkage cracking at ages beyond 7-days of age. Analysis of AASHTO PP34 test results will help determine the exact role of 50% blast furnace slag replacement in shrinkage cracking.

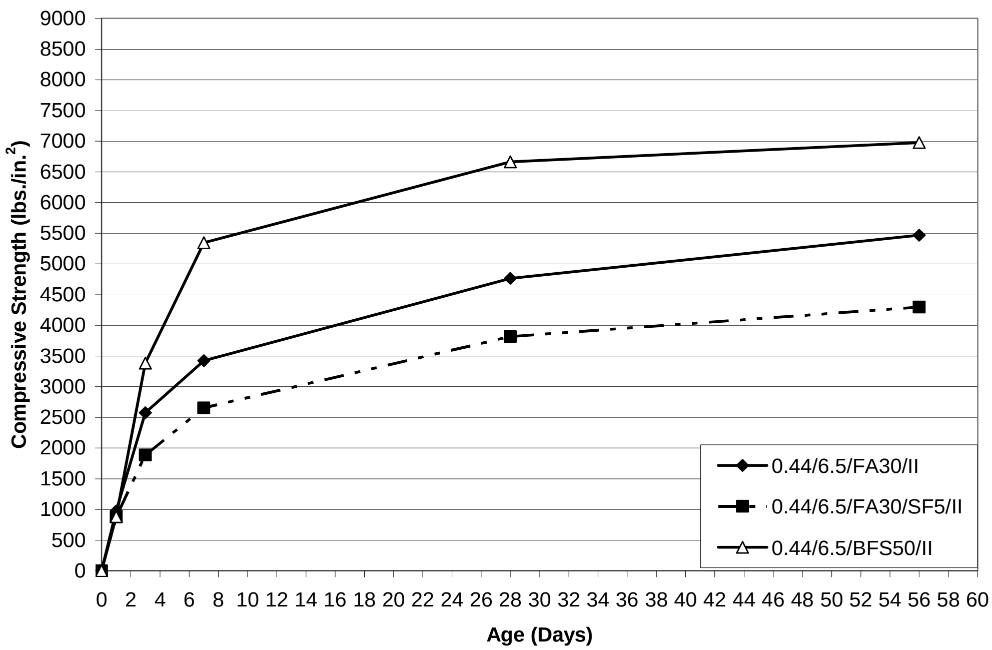

Figure 7.18 Compressive Strength, Mixture #5 (0.44-6.5-FA30-II), Mixture #6 (0.44-6.5-FA30-SF5-II), and Mixture #7 (0.44-6.5-BFS50-II), (ASTM C 39, AASHTO T 22)

7.3.1.5.3 Chemical Admixtures

Beyond 7 days of age the rate of strength gain is very close between Mixture #8 and #9. See Figure 7.19. The set retarder only retarded hydration during the beginning hours of placement and then the rate of strength gain appears to have recovered.

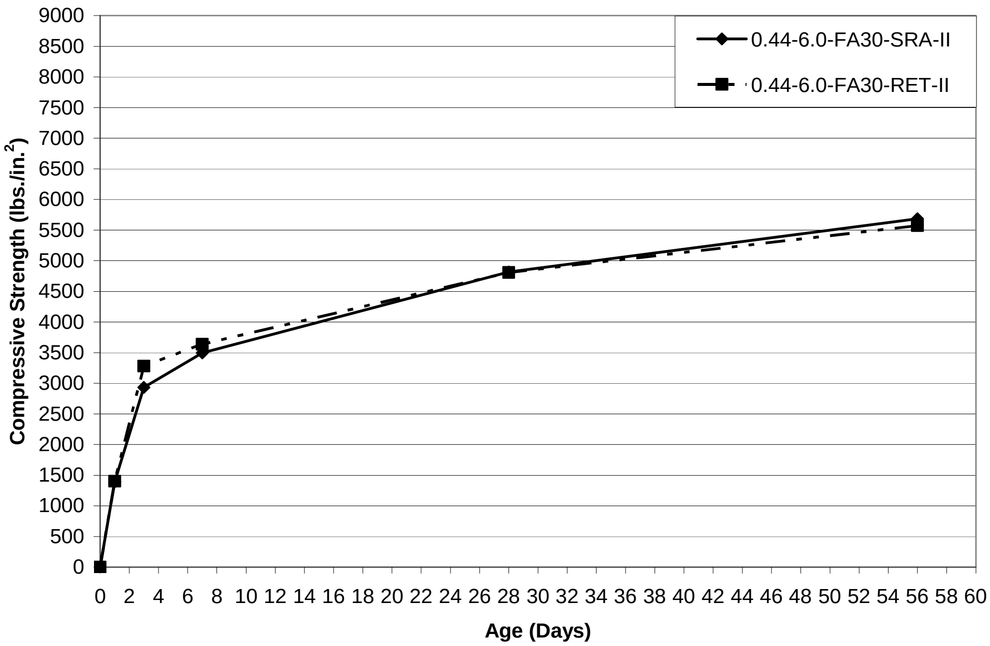

Figure 7.19 Compressive Strength, Mixture #8 (0.44-6.0-FA30-SRA-II) (Shrinkage Reducing Admixture) and Mixture #9 (0.44-6.0-FA30-RET-II) (Set Retarding Admixture), (ASTM C 39, AASHTO T 22)

At 28-days of age, Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II) achieved almost identical compressive strengths of 4817 vs. 4806psi, respectively. It is evident from the figure above that the rate of strength gain for Mixture #8 begins to increase over that of the set retarder Mixture #9 at this time

The trend continues between the two mixtures at 56-days of age. Mixtures #8 (0.44-6.0-FA30-SRA-II) surpassed the compressive strength of Mixture #9 (0.44-6.0-FA30-RET-II) by 2%, 5685 vs. 5572psi. It is clear from the rate of strength gain results that the set retarder only retards the mixture long enough to allow for placement. The rate of strength gain for the shrinkage reducing admixture initially trailed the set retarder mixture up to 7 days of age, at which time it began to increase. At 1, 3, and 7 days of age the shrinkage reducing mixture was below but within 1, 11, and 4%, respectively. At 28 and 56-days of age the shrinkage reducing mixture surpassed the compressive strength of the set retarder mixture by less than 1% and 1%, respectively.

7.3.1.5.4 Aggregate Type

Beyond 7 days of age the trend in the rate of strength gain is reversed and the NWA mixture begins to trail the LWA mixture, Figure 7.20. By 28-days of age the LWA mixture achieved a 2% higher compressive strength than the normal weight aggregate mixture; 5807 vs. 5678psi, respectively.

When the two mixtures reached 56-days of age the internal curing of the lightweight aggregate mixture hydrated the cement particles beyond the normal weight aggregate mixture, reaching a compressive strength of 6273 vs. 5869psi respectively. The continued hydration resulted in a 6% increase in strength by 56-days of age.

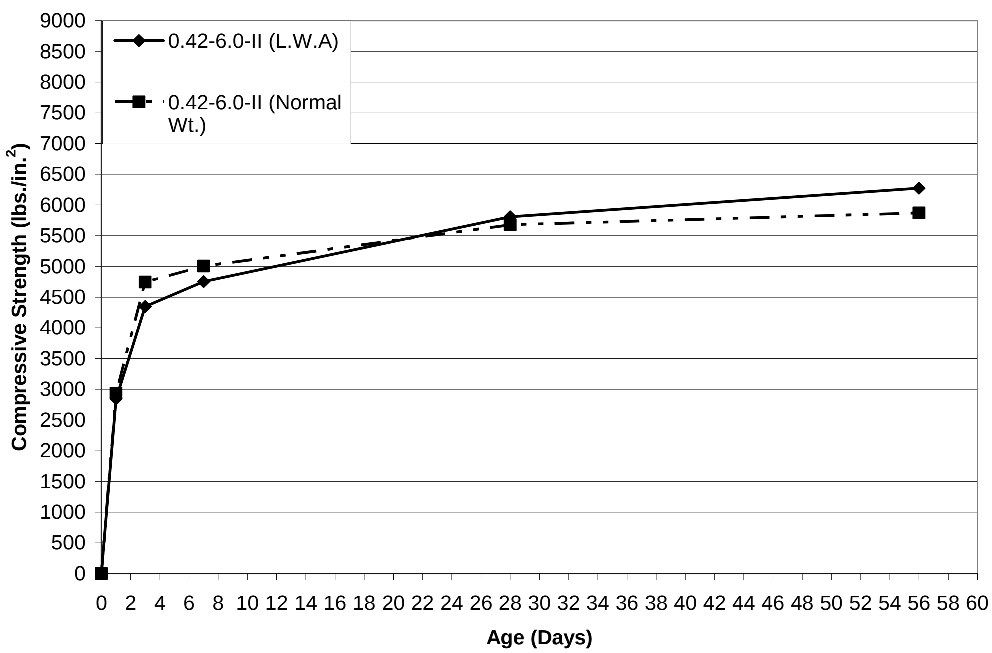

Figure 7.20 Compressive Strength, Mixture #10 (0.42-6.0-II-Lightweight Aggregate) and Mixture #11 (0.42-6.0-II-Normal Weight Aggregate), (ASTM C 39, AASHTO T 22)

7.3.2 Permeability

7.3.2.1 General

The more permeable concrete is the more susceptible it is to damage caused by infiltration of contaminated water. The permeability test performed for this CDOT research is ASTM C 1202 (AASHTO T 227), or the rapid chloride ion penetrability test (RCIP), and was performed at 28 and 56-days of age for each mixture.
Section 3 of ASTM C 1202 summarizes this method as monitoring the amount of electrical current passed through 2-inch (50.8mm) thick slices of 4-inch (101.6mm) nominal diameter cores or cylinders of concrete for a 6 hour period.

The samples were prepared first by wet-saw cutting the top finished surface of a 4" x 8" concrete cylinder specimen. The samples were placed under a dry vacuum (approximately 25 inches (63.5 cm) of mercury) in a desiccator for 3 hours. Water was then introduced to the desiccator and the samples completely submerged. A wet vacuum was pulled for 1 hour before being released. The samples were left to soak in the desiccator, completely submerged in water, for 24 hours, then removed from the water and dried. Silicone was placed around each samples edge to form a seal with a rubber gasket. The cylinder was then placed into the test cell as shown in Figure 7.21.

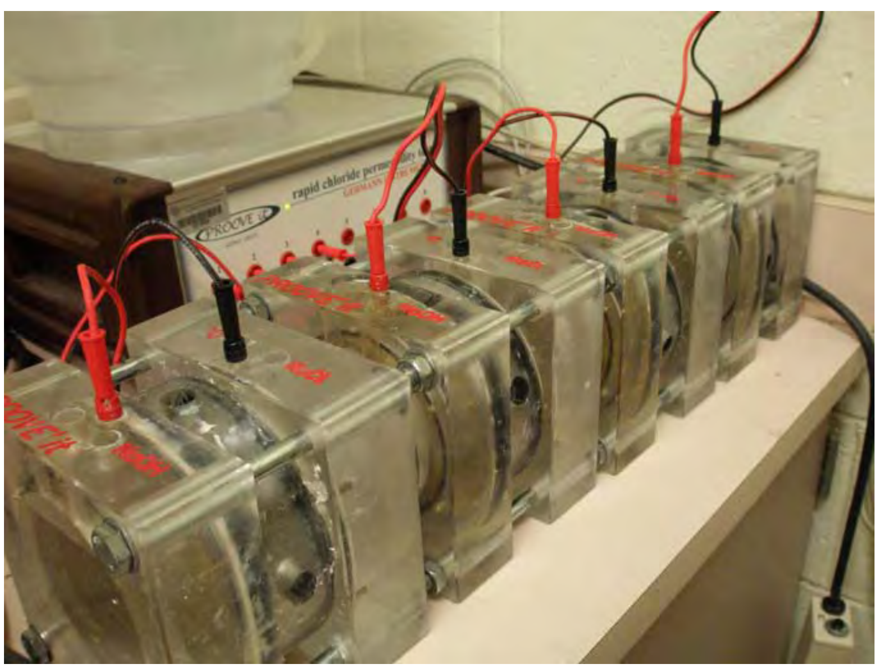

Figure 7.21 Photograph of R.C.I.P. Test Setup

A potential difference of 60-volts (direct-current) is maintained across the ends of the specimen. A sodium chloride solution ($NaCl^-$) fills one side of the apparatus and sodium hydroxide solution ($NaOH^+$) on the other, each saturating its respective end of the sample.

ASTM C 1202 makes a correlation between the total charge passed (coulombs) through the concrete sample and its ability to resist chloride ion penetration. Table 7.4 shows the scale used to designate concretes permeability based upon the coulombs passed.

Table7.4 Permeability Rating per Coulombs Passed

Charge Passed	**Chloride Ion Penetrability**
(Coulombs)	**(Classification)**
> 4000	High
2,000 – 4000	Moderate
1,000 - 2,000	Low
100 - 1,000	Very Low
<100	Negligible

7.3.2.2 Rapid Chloride Ion Penetrability Test

The permeability of concrete develops at various rates and to different magnitudes depending upon the w/cm, cementitious content, and quantity and types of SCMs it contains. Current CDOT Class H and HT specifications require the 56-day permeability not to exceed 2,000 coulombs, or a chloride ion penetrability rating of “low." The results for all eleven mixtures are shown in Table 7.5. Figure 7.21 is a comparison of 28 and 56-day permeability.

Table 7.5 Rapid Chloride Ion Penetrability Results (ASTM C 1202, AASHTO T 227)

Mixture Identification	28-day (coulombs)	Chloride Ion Penetrability	56-day (coulombs)	Chloride Ion Penetrability
0.38/6.8/FA20/SF5/II	685	Very Low	596	Very Low
0.42/6.2/FA16/SF3.5/II	1038	Low	835	Very Low
0.38/6.8/FA20/SF5/G	873	Very Low	373	Very Low
0.42/6.2/FA16/SF3.5/G	3439	Moderate	1965	Low
0.44/6.5/FA30/II	2933	Moderate	1789	Low
0.44/6.5/FA30/SF5/II	2163	Moderate	1387	Low
0.44/6.5/BFS50/II	1272	Low	991	Very Low
0.44-6.0-FA30-SRA-II	2329	Moderate	1400	Low
0.44-6.0-FA30-RET-II	3715	Moderate	1622	Low
0.42-6.0-II (L.W.A)	2396	Moderate	1529	Low
0.42-6.0-II (Normal Wt.)	2100	Moderate	1487	Low

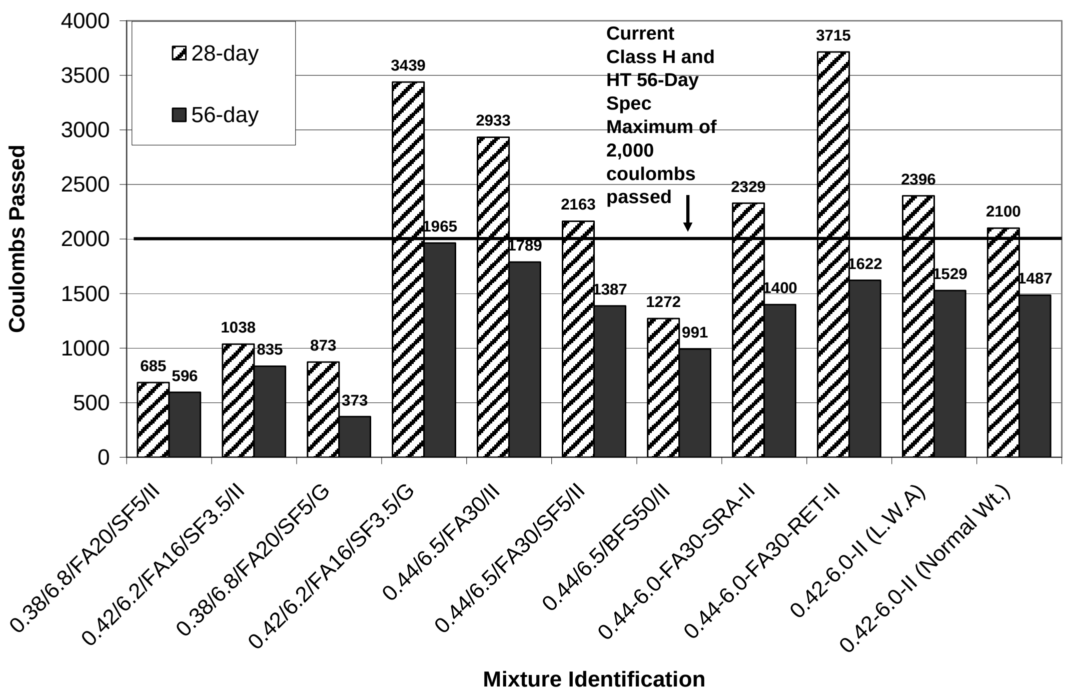

Figure 7.22 Rapid Chloride Ion Penetrability Test Results (Permeability, ASTM C 1202, AASHTO T 227)

All eleven design mixtures exceeded current CDOT Class H and HT requirements of ‘low’ permeability at 56-days of age. This requires fewer than 2,000 coulombs to pass at 56-days of age. Figure 7.23 is a comparison of 56 day permeability.

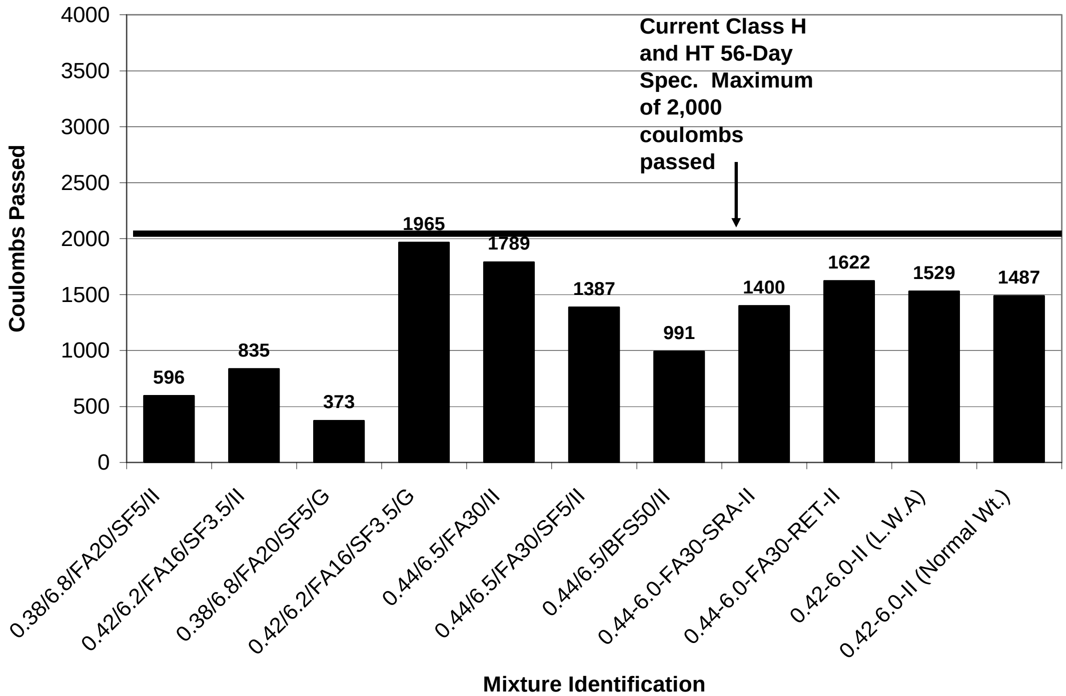

Figure 7.23 56-Day Rapid Chloride Ion Penetrability Test Results (Permeability, ASTM C 1202, AASHTO T 227)

7.3.2.2.1 Cement Type

Mixture #1 (0.38-6.8-FA20-SF5-II), and #3 (0.38-6.8-FA20-SF5-G) have identical mixture proportions and w/cm equal to 0.38, but each is made using a different type of cement (Type II vs. Class G, coarse-ground, respectively). At 28-days of age, the Type G cement mixture is more permeable than the Type II mixture, allowing 27% more coulombs to pass during testing (873 vs. 685 coulombs, Figure 7.24. The rate of hydration of the Type G, coarse-ground cement results in a slight change in the development of permeability. By 56-days of age, the Type G cement concrete mixture began to more rapidly hydrate and the mixture was no longer more permeable than the Type II, but less permeable, allowing 40% fewer coulombs to pass during testing than the Type II mixture (373 vs. 596 coulombs). At a lower w/cm equal to 0.38, the Type G, coarse-ground cement concrete mixtures developed a slightly higher permeability than Type II mixtures at 28-days of age and then lower permeability at 56-days of age. The coarse-ground particles are hydrating more slowly than Type II cement at early ages and more rapidly than Type II cement with increased age. This contrast is evident by the drastic change in the number of coulombs passed by the coarse ground cement mixture from 28 to 56-days of age.

Mixture #3 (w/cm = 0.38), made using Type G cement, showed a decrease in permeability (coulombs passed) by 30% from 28 to 56-days of age while Mixture #1 (0.38-6.8-FA20-SF5-II) (same proportions but made Type II cement) only decreased by 13% during the same period of time. This is seen again by comparing the other two mixtures having identical mixture proportions and higher w/cm with only cement type as a variable. Mixture #4 (w/cm = 0.42), made using Type G cement, showed a decrease in permeability (coulombs passed) by 43% from 28 to 56-days of age while Mixture #2 (0.42-6.2-FA16-SF3.5-II) (same proportions but made suing Type II cement) only decreased by 20% during the same period of time. This data shows Type G, coarse-ground cement reduces concrete permeability more rapidly than Type II cement at later ages.

Mixtures #2 and #4 represent current CDOT Class H and HT specifications having the maximum allowable w/cm equal to 0.42 and lowest allowable replacement percentage of cementitious materials; 16% fly ash, 3.5% silica fume. At 28-days of age, the Type G cement mixture is more permeable than the Type II mixture, allowing 70% more coulombs to pass during testing (3439 vs. 1038 coulombs). The slower hydration rate of the coarse-ground Type G cement particles in Mixture #4 (0.42-6.2-FA16-SF3.5-G) coupled with the increased mix water results in a drastic change in the development of permeability. By 56-days of age, the Type G cement mixture is still more permeable than the Type II, allowing 57% more coulombs to pass during testing than the Type II mixture (1965 vs. 835 coulombs). It must be noted that the air content of Mixture #4 is considerably higher than Mixture #2, thus accounting for the increased permeability. However, Mixture #4 meets the current CDOT requirement for rapid chloride ion penetrability. The Type G cement mixture began with a moderate permeability rating and fell to a low permeability rating by 56-days of age. When the w/cm was increased from

0.38 to 0.42, Type G cement concrete mixtures show a much higher permeability than Type II, cement concrete mixtures at both 28 and 56-days of age.

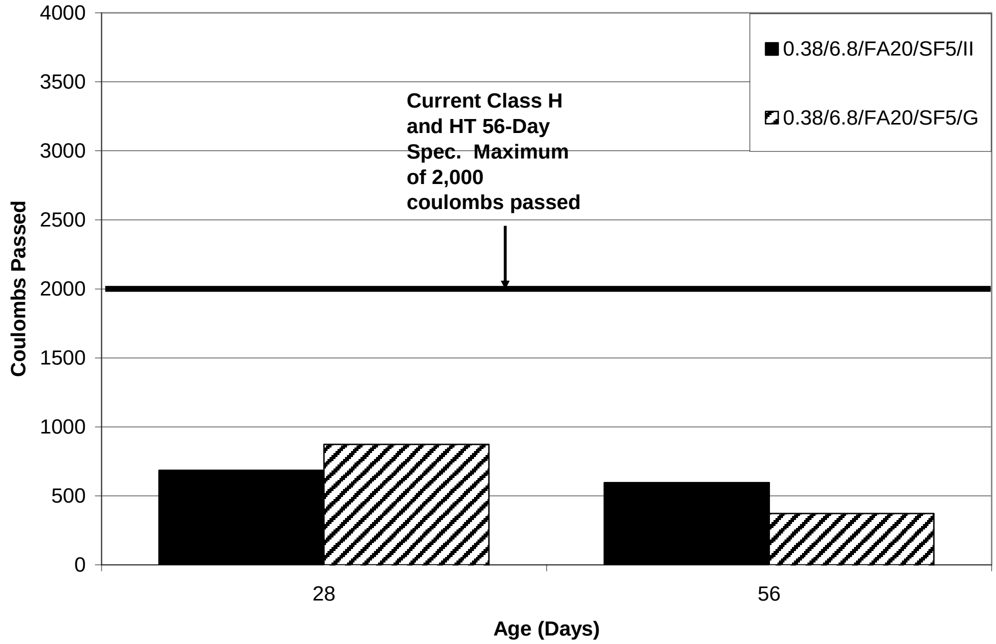

Figure 7.24 28-Day and 56-Day Rapid Chloride Ion Penetrability Test Results, CDOT Control Mixture #1 (0.38-6.8-FA20-SF5-II) (Type II Cement) and Mixture #3 (0.38-6.8-FA20-SF5-G) (Type G, Coarse-Ground Cement) (Permeability, ASTM C 1202, AASHTO T 227)

Mixtures #2 and #4 have the highest w/cm per CDOT Class H and HT specification and the lowest percentage silica fume and fly ash replacement. This combination results in a mixture that is more permeable when compared to Mixture #1 (0.38-6.8-FA20-SF5-II), and #3 (0.38-6.8-FA20-SF5-G), which have the lowest w/cm per the CDOT specifications and the highest percentage silica fume and fly ash replacement. A combination of more silica fume and a lower w/cm typically result in a less permeable concrete, as seen by the results.

Mixture #4 (0.42-6.2-FA16-SF3.5-G) has the highest permeability of all the mixtures batched thus far. This mixture has w/cm equal to 0.42 but has the lowest allowable percentage cementitious materials replacement allowed per CDOT specifications.

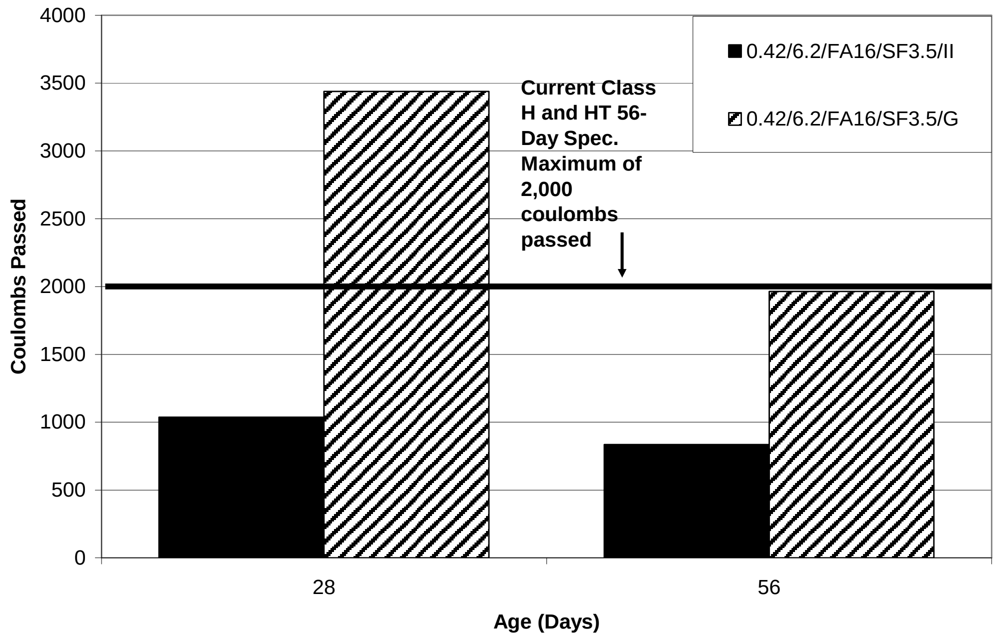

Figure 7.25 28-Day and 56-Day Rapid Chloride Ion Penetrability Test Results, CDOT Control Mixture #2 (0.42-6.2-FA16-SF3.5-II) (Type II Cement) and Mixture #4 (0.42-6.2-FA16-SF3.5-G) (Type G, Coarse-Ground Cement), (Permeability, ASTM C 1202, AASHTO T 227)

7.3.2.2.2 Supplementary Cementitious Materials

Mixtures #5, #6, and #7 experienced moderate permeability at 28-days of age. This is significantly higher than the first three mixtures. This trend is due to higher w/cm ratios equal to 0.44 versus w/cm equal to 0.38 and 0.42 for Mixtures #1-#4. The increased mix water resulted in increased permeability. However, the 56-day permeability results satisfy the CDOT specifications. See Figure 7.26.

Mixture #5 (0.44-6.5-FA30-II) and Mixture #6 (0.44-6.5-FA30-SF5-II) have the same w/cm and fly ash replacement but Mixture #6 introduces a 5% replacement with silica fume. This explains the decreased permeability (coulombs passed) at 28-days of age; 2163 to 2933 coulombs, respectively. There is a decrease of 26% due to the 5% silica fume replacement. By 56-days of age the silica fume in Mixture #6 (0.44-6.5-FA30-SF5-II) decreased the concrete permeability by 23% compared to the fly ash Mixture #5 (0.44-6.5-FA30-II), 1387 vs. 1789 (coulombs passed). Mixture #5 showed a

39% decrease in permeability from 28 to 56-days of age, while Mixture #6 showed a similar decrease of 36%. The silica fume hydrated primarily during the first 28-days of age, resulting in a more substantial decrease in permeability. As a result, slightly less water remained for continued hydration of the cement particles beyond 28-days; slowing the rate of impermeability. The mixture made without silica fume had a more even distribution of water for the hydration of cement particles over time.

Mixture #7 (0.44-6.5-BFS50-II), although designed with an increased w/cm equal to 0.44, exhibited 'low' permeability at 28-days of age due to the 50% replacement of cement with blast furnace slag (1272 coulombs passed). This replacement decreased the concretes permeability at 56-days of age to a rating of 'very low' (991 coulombs passed). This is a 22% decrease in permeability from 28 to 56-days of age and is less than Mixture #5 (0.44-6.5-FA30-II) and Mixture #6 (0.44-6.5-FA30-SF5-II) made using silica fume and Class F fly ash replacement of cement; 39% and 36% respectively.

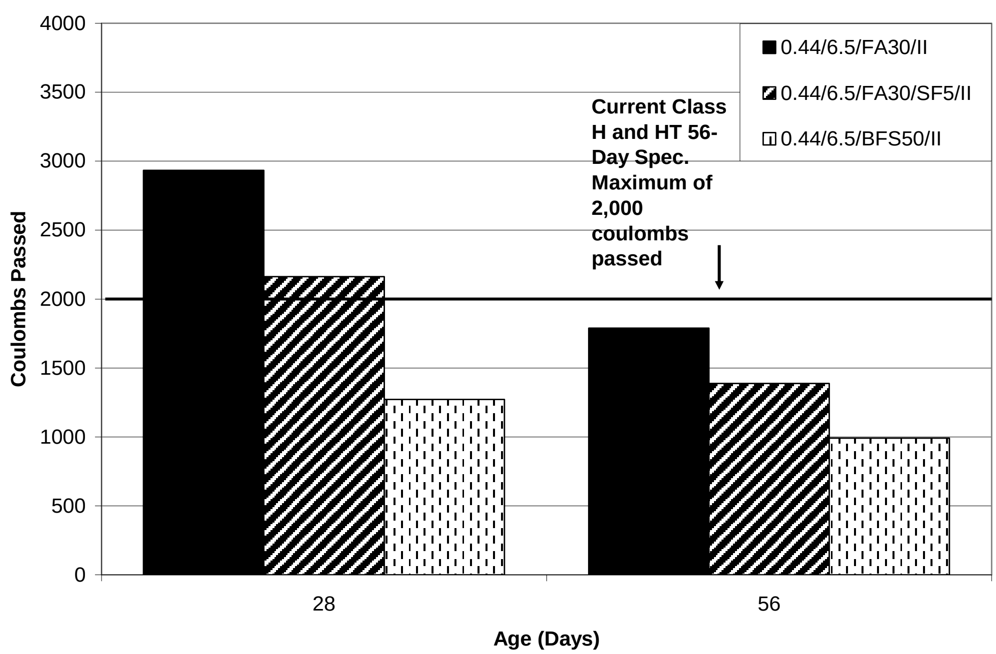

Figure 7.26 28-Day and 56-Day Rapid Chloride Ion Penetrability Test Results, Mixture #5 (0.44-6.5-FA30-II), Mixture #6 (0.44-6.5-FA30-SF5-II), and Mixture #7 (0.44-6.5-BFS50-II) (Permeability, ASTM C 1202, AASHTO T 227)

7.3.2.2.3 Chemical Admixtures

Mixture #8 (0.44-6.0-FA30-SRA-II-Shrinkage Reducing Admixture) and Mixture #9 (0.44-6.0-FA30-RET-II-Set Retarding Admixture) were batched at the same time. Both the set retarder mixture and the shrinkage reducing mixture had a w/cm equal to 0.44 and developed 'moderate' permeability by 28-days of age. Although they have the same water content and are in the same category at 28-days of age, Mixture #9 (0.44-6.0-FA30-RET-II) developed 37% lower permeability than Mixture #8 (0.44-6.0-FA30-SRA-II); 3715 vs. 2329 coulombs passed, respectively. The permeability for the set retarder mixture decreased 44% between 28 and 56-days of age, while the shrinkage reducing mixture decreased 60%.

As previously discussed, the set retarder allows for a slower initial hydration of the cement. Typically, slower initial hydration will result in increased long-term strength and reduced permeability. The admixture also retards the rate of permeability decrease up to 28-days of age. After 28-days of age the concrete's permeability decreases at an increased rate.

7.3.2.2.4 Aggregate Type

Mixture #10 (0.42-6.0-II-Lightweight Aggregate) and Mixture #11 (0.42-6.0-II-Normal Weight Aggregate) have similar fresh and hardened concrete properties up to 28-days of age. It is expected for the LWA mixture to have a higher permeability at 28-days. However, the additional hydration (internal curing) provided from the LWA is expected to decrease permeability at ages beyond 28-days.

At 28-days of age, Mixture #10 (0.42-6.0-II-Lightweight Aggregate) developed 12% lower permeability than Mixture #11 (0.42-6.0-II-Normal Weight Aggregate); 2396 vs. 2100 coulombs passed. These results classify the two mixtures as having 'moderate' permeability at 28-days of age. At 28-days of age, Mixture #10 (0.42-6.0-II-Lightweight Aggregate) and Mixture #11 (0.42-6.0-II-Normal Weight Aggregate) are each within 15% and 5% of meeting the CDOT Class H and HT specification, respectively.

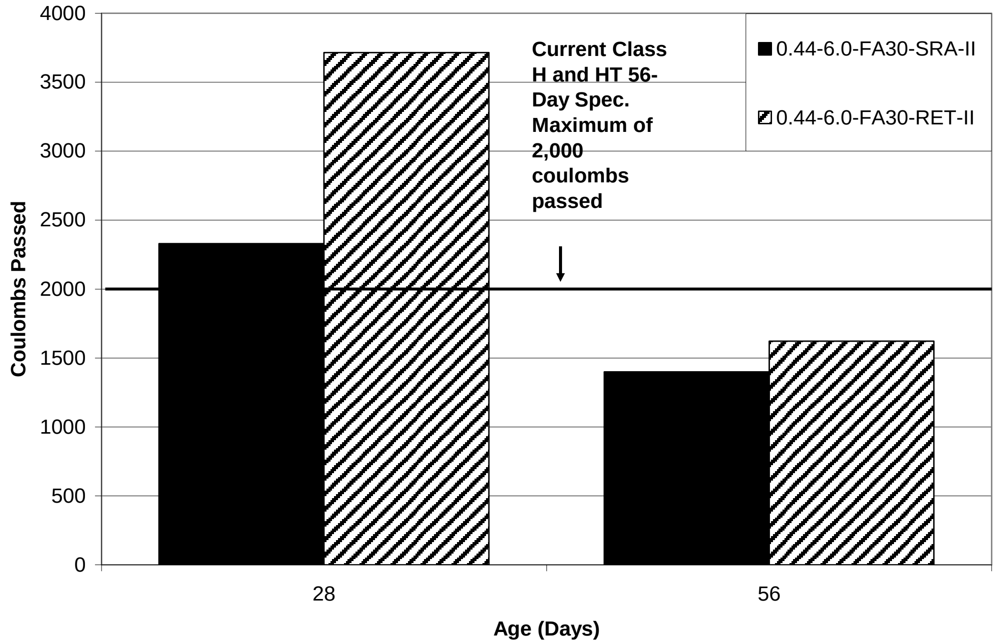

Figure 7.27 28-Day and 56-Day Rapid Chloride Ion Penetrability Test Results, Mixture #8 (0.44-6.0-FA30-SRA-II) (Shrinkage Reducing Admixture) and Mixture #9 (0.44-6.0-FA30-RET-II) (Set Retarding Admixture) (Permeability, ASTM C 1202, AASHTO T 227)

By 56-days of age, both mixtures easily exceed the current specification. Mixture #10 (0.42-6.0-II-Lightweight Aggregate) and Mixture #11 (0.42-6.0-II-Normal Weight Aggregate) surpassed the specification by 24 and 26% respectively. See Figure 7.28. The LWA mixture experienced a significant decrease in permeability between 28 and 56 days due to the continued hydration provided by the additional moisture in the aggregate.

7.3.3 Durability

7.3.3.1 General

Concrete's permeability provides an indication of its ability to resist or allow water to enter. When water freezes it expands by volume. The more permeable concrete is the more water it will allow to penetrate. When water is allowed to penetrate and freezing temperatures (cycles) occur, the water freezes inside the concrete and expands against the rigidity of the concrete. The volume expansion of the water creates internal stresses that are damaging to the concrete. Air voids within the concrete structure alleviate the stresses caused by this volume expansion.

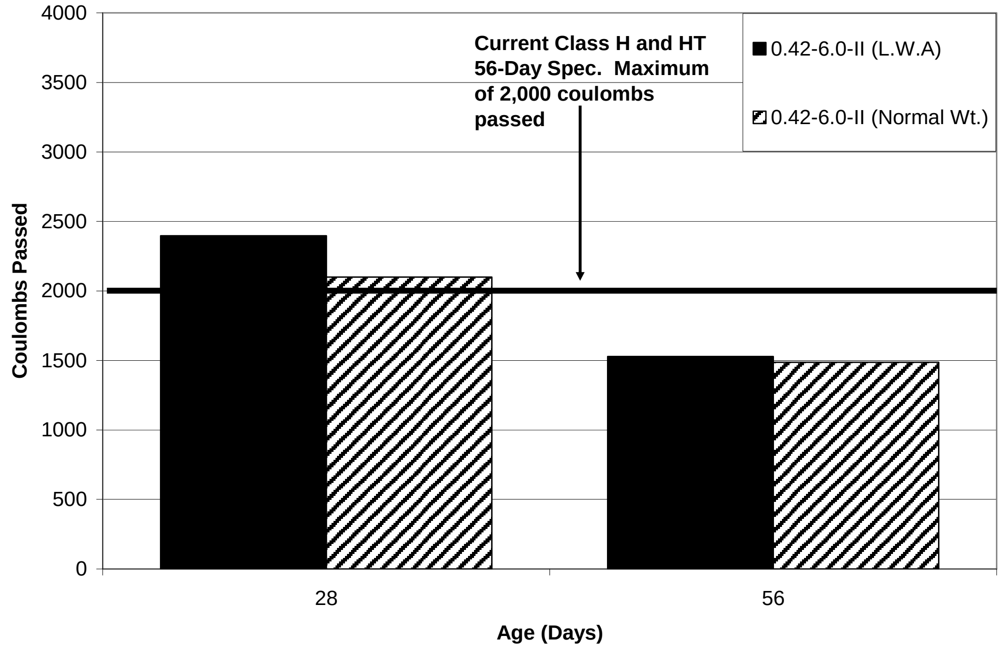

Figure 7.28 28-Day and 56-Day Rapid Chloride Ion Penetrability Test Results, Mixture #10 (0.42-6.0-II-Lightweight Aggregate) and Mixture #11 (0.42-6.0-II-Normal Weight Aggregate) (Permeability, ASTM C 1202, AASHTO T 227)

Depending upon the climate, concrete is designed to contain air voids (air content %) to enhance its durability. Increased air content will improve the durability of concrete in areas like Colorado, where freezing temperatures occur more often or for longer periods of time. As a result, it is of interest to research the durability of concrete proposed for use in Colorado bridge decks and exposed to freeze/thaw conditions. The ability of concrete to resist freeze/thaw cycles translates to durability. A more durable concrete will better resist the harmful effects caused by freeze/thaw cycles. The freeze/thaw resistance test chosen for this research is the ASTM C 666 Procedure A (AASHTO T 161). Figure 7.29 is a photograph of the University of Colorado Denver, Material's Testing Laboratory, freeze/thaw chamber.

Figure 7.29 Photograph of Freeze/Thaw Chamber (ASTM C 666, Procedure A)

Two freeze/thaw beams were fabricated for each of the eleven mixtures batched during this study. The beams were cured until 14 days of age. At 14 days of age the beams were removed from the curing tank and weighed, and their initial resonant frequencies measured per ASTM C 666 prior to being subjected to any freeze/thaw cycles. The beams were then placed in individual metal holding containers in the freeze/thaw chamber. Each container was filled with water to completely submerge the beam and freeze/thaw cycles ensued.

The performance of the specimens during the freeze/thaw testing was determined by measuring each specimen's resonant frequency. Two methods of determining the specimen's resonant frequencies included static and dynamic testing procedures. Both methods meet the ASTM 666 standard. After testing, the beams were placed back in the freeze/thaw chamber for approximately 28 additional freeze/thaw cycles. The chamber simulates approximately four, six-hour cycles per day (0 to – 40^{o}F or -17^{o} to 4^{o}C) producing 28 cycles every 7 days. After 28 cycles the beams were removed, weighed, and their resonant frequencies measured again. Figures 7.30 and 7.31 are photographs of the E-meter and the static durability test apparatus.

Figure 7.30 Photograph of Durability Testing Apparatus (ASTM C 666, Procedure A)

Figure 7.31 Photograph of Durability Testing Apparatus (ASTM C 666, Procedure A)

A transmitter sends a frequency from the mid-section of the beam and a receiver at the left-end of the beam receives the frequency. With exposure to freezing and thawing the beam develops cracks and voids internally when the water expands. The more cracks or voids within the beam the more frequency that is lost in transmission and unable to be received at the end of the beam. As the beam deteriorates and more cracks occur inside, the resonant frequency diminishes. Using the measured resonant frequency and the corresponding number of freeze/thaw cycles the beam has been exposed to, two calculations are possible. The relative dynamic modulus of elasticity and the durability factor are values used to describe the durability of concrete. Results from the eleven mixtures are shown in the Tables 7.6 – 7.16.

Table 7.6 Mixture #1 (0.38-6.8-FA20-SF5-II), Freeze/Thaw Results

	Specimen A		Specimen B	
Cycles	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	1992	2029	2031	2066
28	1914	1989	1953	1997
56	1914	1963	1953	1947
84	1895	1984	1953	1963
112	1953	1919	1973	1957
140	1914	1906	1973	1934
168	1973	1970	2012	2026
196	1973	1999	2012	1987
224	1973	1971	1992	2030
252	1934	1984	1953	1990
280	1934	1931	1953	1983
316	1992	2008	1992	2017

Table 7.7 Mixture #2 (0.42-6.2-FA16-SF3.5-II), Freeze/Thaw Results

	Specimen A		Specimen B	
Cycles	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	1914	1905	1914	1910
28	1836	1850	1875	1859
56	1823	1850	1836	1855
84	1836	1863	1855	1854
112	1855	1841	1875	1847
140	1823	1835	1856	1842
168	1895	1888	1895	1912
196	1895	1872	1914	1874
224	1895	1869	1875	1908
252	1855	1852	1855	1842
280	1850	1855	1875	1855
316	1914	1923	1895	1910

Table 7.8 Mixture #3 (0.38-6.8-FA20-SF5-G), Freeze/Thaw Results

Cycles	Specimen A Dynamic (Hz)	Specimen A Static (Hz)	Specimen B Dynamic (Hz)	Specimen B Static (Hz)
0	2188	2194	2188	2204
28	2090	2111	2090	2099
56	2051	2108	2038	2041
84	1061	2098	990	2030
112	2012	2071	1962	2045
140	1992	2052	1992	2011
168	1973	2050	1986	2006
196	1973	2044	1986	1998
224	1927	2009	1921	1972
252	1934	2003	938	1933
280	1914	1997	1914	1929
308	1953	2029	1901	1968

Table 7.9 Mixture #4 (0.42-6.2-FA16-SF3.5-G), Freeze Thaw Results

Cycles	Specimen A		Specimen B	
	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	1934	1945	1934	1948
28	1855	1894	1836	1881
36	1914	1925	1914	1910
78	1895	1909	1875	1886
116	1836	1889	1836	1878
134	1875	1888	1836	1878
162	1875	1890	1855	1877
190	1855	1894	1836	1881
220	1875	1921	1855	1896
253	1816	1861	1797	1839
283	1875	1895	1836	1877
313	1875	1871	1836	1863

Table 7.10 Mixture #5 (0.44-6.5-FA30-II), Freeze/Thaw Results

Cycles	Specimen A		Specimen B	
	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	2051	2013	2051	2031
28	1973	2002	1914	1952
36	2031	2040	1992	2008
78	2012	2025	1992	2003
116	1953	1988	1934	1981
134	1953	1968	1934	1945
162	1973	1977	1934	1963
190	1934	1947	1875	1898
220	1934	1982	1875	1946
253	1895	1925	1855	1900
283	1914	1912	1875	1895
313	1914	1922	1875	1880

Table 7.11 Mixture #6 (0.44-6.5-FA30-SF5-II), Freeze/Thaw Results

	Specimen A		Specimen B	
Cycles	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	1934	1950	1934	1946
42	1895	1889	1875	1894
78	1836	1857	1836	1865
98	1816	1849	1855	1886
126	1836	1860	1875	1889
154	1758	1788	1816	1850
184	1836	1850	1836	1867
217	1738	1786	1758	1797
247	1816	1830	1836	1843
277	1816	1808	1797	1809
308	1797	1840	1758	1857

Table 7.12 Mixture #7 (0.44-6.5-BFS50-II), Freeze/Thaw Results

Cycles	Specimen A		Specimen B	
	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	2090	2112	2168	2176
42	1992	2012	2070	2097
78	1934	1979	2012	2040
98	1927	1979	1953	2013
126	1875	1910	1953	1985
154	1777	1870	1816	1865
184	1797	1869	1914	1977
217	1680	1763	1758	1858
247	1797	1845	1914	1929
277	1758	1760	1855	1875
308	1758	1752	1855	1839

Table 7.13 Mixture #8 (0.44-6.0-FA30-SRA-II), Freeze/Thaw Results

Cycles	Specimen A		Specimen B	
	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	1465	1456	1504	1467
30	1313	1297	1341	1323
60	1087	1086	1133	1111
100	957	956	918	945

Table 7.14 Mixture #9 (0.44-6.0-FA30-RET-II), Freeze/Thaw Results

Cycles	Specimen A		Specimen B	
	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	1973	1972	1973	1962
30	1973	1960	1957	1950
60	1992	1999	1992	1977
100	1934	1930	1953	1933
140	1992	N/A	1992	N/A
180	1973	1934	1957	1952
210	1953	1956	1934	1942
250	1914	1933	1875	1836
290	1972	1941	1953	1936
330	1934	1912	1914	1936

Table 7.15 Mixture #10 (0.42-6.0-II-Lightweight Agg.), Freeze/Thaw Results

	Specimen A		Specimen B	
Cycles	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	2051	2048	2011	2011
40	1973	1960	1953	1947
80	1962	N/A	1953	N/A
120	1933	1968	1933	1955
150	1972	1958	1972	1933
190	1933	1934	1894	1895
230	1953	1969	1914	1921
270	1933	1896	1914	1912
310	1914	1895	1894	1895

Table 7.16 Mixture #11 (0.42-6.0-II-Normal Weight Agg.), Freeze/Thaw Results

Cycles	Specimen A		Specimen B	
	Dynamic (Hz)	Static (Hz)	Dynamic (Hz)	Static (Hz)
0	2109	2100	2128	2123
40	2051	2018	2070	2050
80	2051	N/A	2051	N/A
129	2041	2040	2051	2054
150	2031	2024	2041	2057
190	1953	1948	1972	1969
230	2011	1995	1992	1970
270	1972	1985	1972	1973
310	1953	1953	1914	1961

The relative dynamic modulus of elasticity (P_c), used to calculate the durability factor, is a ratio of the initial frequency (n) to the frequency when the test is terminated (n_1). The test ends after 300 freeze/thaw cycles or when the relative modulus of elasticity of the test specimen has diminished to 60% of the initial modulus (the modulus prior to freeze/thaw exposure). Calculation of the relative modulus of elasticity is performed using Equation 5.

$$P_c = (n_1^2 / n^2) \times 100$$ **Equation 5**

where:

Pc = relative dynamic modulus of elasticity, after c cycles of freezing and thawing, percent,

n = fundamental transverse frequency at 0 cycles of freezing and thawing

n_1 = fundamental transverse frequency after c cycles of freezing and thawing

Note 9 of ASTM C 666 states: This calculation of relative dynamic modulus of elasticity is based on the assumption that the mass and dimensions of the specimen remain constant throughout the test. This assumption is not true in many cases due to disintegration of the specimen. However, if the test is to be used to make comparisons between the relative dynamic modulus of different specimens or of different concrete formulations, P_c as defined is adequate for the purpose.

The durability factor (DF) is a ratio of the number of cycles at test termination (N) to the number of cycles when the test is to be terminated (M) and is equal to 300 cycles. This ratio is multiplied by the relative dynamic modulus, P_c (%), at N cycles. Calculation of the durability factor is performed using Equation 6.

$$DF = (P \times N) / M$$ **Equation 6**

where:

DF = durability factor of the test specimen,

P = relative dynamic modulus of elasticity at N cycles, %,

N = number of cycles at which P reaches the specified minimum value for discontinuing the test or the specified number of cycles at which the exposure is to be terminated, whichever is less, and

M = specified number of cycles at which the exposure is to be terminated

The relative dynamic modulus of elasticity for both methods is shown in Tables 7.17-7.27 below.

Table 7.17 Mixture #1 (0.38-6.8-FA20-SF5-II), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	2012	100.00	2048	100.00
28	1934	92.38	1993	94.75
56	1934	92.38	1955	91.17
84	1924	91.45	1974	92.90
112	1963	95.20	1938	89.59
140	1943	93.32	1920	87.93
168	1992	98.07	1998	95.22
196	1992	98.07	1993	94.75
224	1982	97.11	2001	95.46
252	1943	93.32	1987	94.18
280	1943	93.32	1957	91.36
316	1992	98.07	2013	96.61

Table 7.18 Mixture #2 (0.42-6.2-FA16-SF3.5-II), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	1914	100.0	1908	100.0
28	1855	94.0	1855	94.5
56	1829	91.4	1853	94.3
84	1846	93.0	1859	94.9
112	1865	95.0	1844	93.5
140	1839	92.3	1839	92.9
168	1895	98.0	1900	99.2
196	1904	99.0	1873	96.4
224	1885	97.0	1889	98.0
252	1855	94.0	1847	93.8
280	1855	94.0	1871	96.2
316	1904	99.0	1917	100.9

Table 7.19 Mixture #3 (0.38-6.8-FA20-SF5-G), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	2188	100.0	2199	100.0
28	2090	91.3	2105	91.6
56	2044	87.3	2075	89.0
84	1025	22.0	2064	88.1
112	1987	82.5	2058	87.6
140	1992	82.9	2032	85.3
168	1979	81.9	2028	85.1
196	1980	81.9	2021	84.5
224	1924	77.3	1991	81.9
252	1436	43.1	1968	80.1
280	1914	76.6	1963	79.7
308	1927	77.6	1999	82.6

Table 7.20 Mixture #4 (0.42-6.2-FA16-SF3.5-G), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	1934	100.0	1947	100.0
28	1846	91.1	1888	94.0
36	1914	98.0	1918	97.0
78	1885	95.0	1898	95.0
116	1836	90.2	1884	93.6
134	1855	92.1	1883	93.6
162	1865	93.1	1884	93.6
190	1846	91.1	1888	94.0
220	1865	93.1	1909	96.1
253	1807	87.3	1850	90.3
283	1855	92.1	1886	93.9
313	1855	92.1	1867	92.0

Table 7.21 Mixture #5 (0.44-6.5-FA30-II), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	2051	100.0	2022	100.0
28	1943	89.8	1977	95.6
36	2012	96.2	2024	100.2
78	2002	95.3	2014	99.2
116	1943	89.8	1985	96.3
134	1943	89.8	1957	93.6
162	1953	90.7	1970	94.9
190	1904	86.2	1923	90.4
220	1904	86.2	1964	94.3
253	1875	83.6	1913	89.5
283	1895	85.3	1904	88.6
313	1895	85.3	1901	88.4

Table 7.22 Mixture #6 (0.44-6.5-FA30-SF5-II), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	1934	100.0	1948	100.0
42	1885	95.0	1892	94.3
78	1836	90.2	1861	91.3
98	1836	90.2	1868	91.9
126	1855	92.1	1875	92.6
154	1787	85.4	1819	87.2
184	1836	90.2	1859	91.0
217	1748	81.7	1792	84.6
247	1826	89.2	1837	88.9
277	1807	87.3	1809	86.2
308	1777	84.5	1849	90.0

Table 7.23 Mixture #7 (0.44-6.5-BFS50-II), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	2129	100.0	2144	100.0
42	2031	91.0	2055	91.8
78	1973	85.9	2010	87.8
98	1940	83.0	1996	86.7
126	1914	80.8	1948	82.5
154	1797	71.2	1868	75.9
184	1855	76.0	1923	80.4
217	1719	65.2	1811	71.3
247	1855	76.0	1887	77.5
277	1807	72.0	1818	71.9
308	1807	72.0	1796	70.1

Table 7.24 Mixture #8 (0.44-6.0-FA30-SRA-II), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	1484	100.0	1462	100.0
30	1327	79.9	1310	80.3
60	1110	55.9	1099	56.5
100	938	39.9	951	42.3

Table 7.25 Mixture #9 (0.44-6.0-FA30-RET-II), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	1973	100.0	1967	100.0
30	1965	99.2	1955	98.8
60	1992	102.0	1988	102.1
100	1943	97.1	1932	96.4
140	1992	102.0	N/A	N/A
180	1965	99.2	1943	97.6
210	1943	97.1	1949	98.2
250	1895	92.2	1885	91.8
290	1963	99.0	1939	97.1
330	1924	95.1	1924	95.7

Table 7.26 Mixture #10 (0.42-6.0-II-Lightweight Agg.), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	2031	100.0	2030	100.0
40	1963	93.4	1954	92.7
80	1958	92.9	N/A	N/A
120	1934	90.6	1962	93.4
150	1973	94.3	1946	91.9
190	1914	88.8	1915	89.0
230	1934	90.6	1945	91.8
270	1924	89.7	1904	88.0
310	1904	87.9	1895	87.2

Table 7.27 Mixture #11 (0.42-6.0-II-Normal Weight Agg.), Relative Dynamic MOE

	Dynamic		Static	
Cycles	Average Frequency	Relative Modulus (%)	Average Frequency	Relative Modulus (%)
0	2119	100.0	2112	100.0
40	2061	94.5	2034	92.8
80	2051	93.7	N/A	N/A
120	2046	93.2	2047	94.0
150	2041	92.8	2041	93.4
190	1963	85.8	1959	86.0
230	2002	89.2	1983	88.2
270	1973	86.7	1979	87.8
310	1934	83.3	1957	85.9

The durability factors for all eleven mixtures are shown in Table 7.28. Mixtures with a durability factor greater than 60 are classified has adequate freeze/thaw resistance. Ten of the eleven mixtures examined in this study exhibited excellent freeze/thaw resistance. Mixture #8 (0.44-6.0-FA30-SRA-II) had decreased freeze/thaw resistance as a result of low air content.

Table 7.28 Durability Factors

Mixture Number	Mixture Identification	# of Cycles	Durability Factor Static	Dynamic	Average	Air Content	*Percent Difference*
						%	
1	0.38/6.8/FA20/SF5/II	316	101.8	103.3	**102.6**	5.5	*1.5%*
2	0.42/6.2/FA16/SF3.5/II	316	106.3	104.3	**105.3**	8.0	*1.9%*
3	0.38/6.8/FA20/SF5/G	308	84.8	79.7	**82.3**	3.4	*6.0%*
4	0.42/6.2/FA16/SF3.5/G	313	96.0	96.1	**96.1**	9.5	*0.1%*
5	0.44/6.5/FA30/II	313	92.2	89.0	**90.6**	4.5	*3.5%*
6	0.44/6.5/FA30/SF5/II	308	92.4	86.7	**89.6**	9.0	*6.2%*
7	0.44/6.5/BFS50/II	308	72.0	73.9	**73.0**	3.5	*2.6%*
8	0.44-6.0-FA30-SRA-II	60	11.3	11.2	**11.3**	2.8	*0.9%*
9	0.44-6.0-FA30-RET-II	330	105.2	104.6	**104.9**	7.5	*0.6%*
10	0.42-6.0-II (L.W.A)	310	90.1	90.8	**90.5**	7.5	*0.8%*
11	0.42-6.0-II (Normal Wt.	310	88.6	86	**87.3**	7.5	*2.9%*

As previously mentioned, air content has a direct effect on the durability of concrete however; air content alone does not provide sufficient durability. As air content in concrete increases, the durability factor increases to a point. As air content becomes too high, the concrete is not strong enough to resist the internal stresses caused by freeze/thaw. It is clear from the data above that air content alone does not control the durability of concrete. Concrete with low air content will deteriorate more quickly and have lower durability than a concrete mixture with increased air contents; however, supplementary cementitious materials contained in the mixture demonstrate a significant impact.

7.3.3.2 Durability Analysis

7.3.3.2.1 Cement Type

The air content for Mixture #1 (0.38-6.8-FA20-SF5-II) and #2 (0.42-6.2-FA16-SF3.5-II) are 5.5 and 8.0%, respectively. The durability factors for the two mixtures are relatively close, 102.6 and 105.3, respectively. See Figure 7.32. The increased air content of Mixture # 2 did increase the durability of the concrete. The w/cm for the two mixtures was 0.38 and 0.42. Mixture #1 had the lowest w/cm and highest percentage of cement replacement by fly ash and silica fume allowed per current CDOT Class H and HT specifications. Mixture #2 had the highest w/cm and lowest allowable replacement percentages. Both mixtures prove to be very durable, maintaining relative moduli well above 60% and resisting over 300 freeze/thaw cycles.

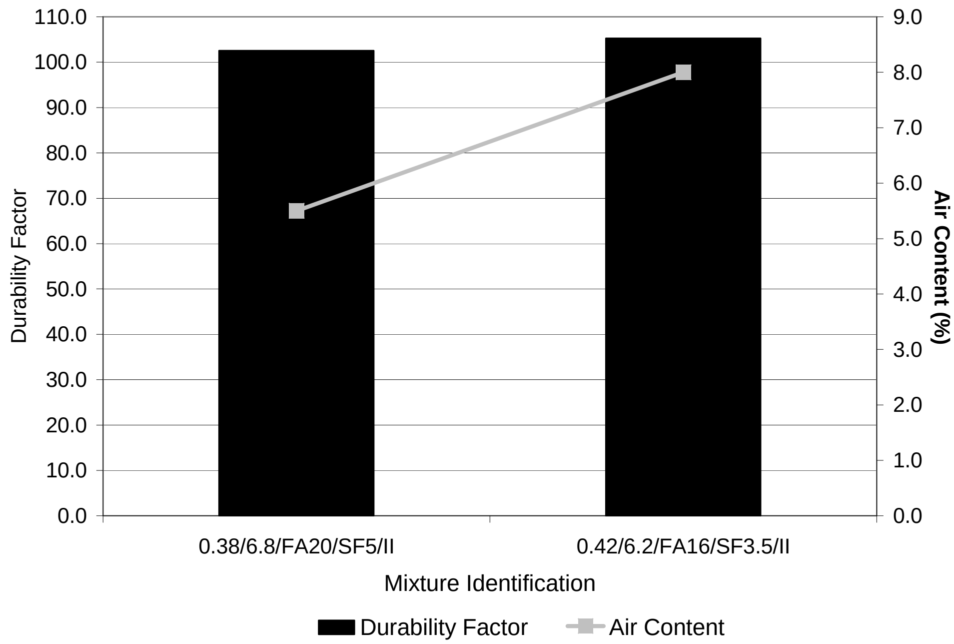

Figure 7.32 Durability Factor and Air Content, CDOT Control Mixture #1 (0.38-6.8-FA20-SF5-II) and CDOT Control Mixture #2 (0.42-6.2-FA16-SF3.5-II)

Mixture #3 (0.38-6.8-FA20-SF5-G) is similar to Mixture #1, but it is made using Type G, coarse ground cement instead of the specified Type II. Mixture #3 had a durability factor 20% greater than Mixture #1, 102.6 vs. 82.3. This is believed to be the result of the coarse ground cement. Furthermore, Mixture #3 experienced significantly greater strength and decreased permeability at 28 and 56 days of age. The increase in freeze/thaw resistance is a function of the decreased permeability.

Mixture #4 (0.42/6.2/FA16/SF3.5/G) counters Mixture #2 (0.42/6.2/FA16/SF3.5/II) but is made using Type G, coarse ground cement instead of Type II. Mixtures #2 and #4 have air contents equal to 8% and 9.5%, respectively. If air content alone affected durability, the 1.5% difference would result in a similar difference of durability factors as seen between Mixture #1 and #3 (2% difference in air content to 20% difference in durability factors). However, Mixture #4 (0.42/6.2/FA16/SF3.5/G) had the higher air content but resulted in lower durability. The w/cm and supplementary cementitious materials replacements for the two mixtures are identical. The durability factor peaks at approximately 9% air content. Any mixtures exceeding such an air content have so much air that the concrete isn't strong enough to resist stresses and weakens the concrete. This is believed to be responsible for the difference in durability between the two mixtures.

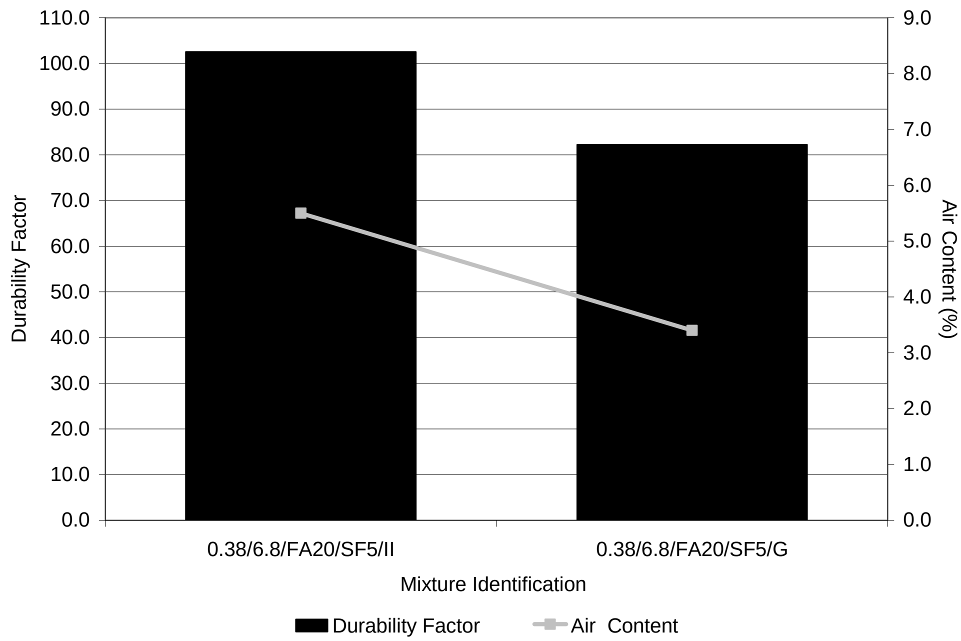

Figure 7.33 Durability Factor and Air Content, CDOT Control Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #3 (0.38-6.8-FA20-SF5-G)

7.3.3.2.2 Supplementary Cementitious Materials

Mixtures #5 (0.44/6.5/FA30/II), #6 (0.44/6.5/FA30/SF5/II), and #7 (0.44/6.5/BFS50/II) all have the same w/cm (0.44) but each introduces various amounts of cement replacement with supplementary cementitious materials; 30% Class F fly ash alone, 30% Class F fly ash with of 5% silica fume, and only 50% blast furnace slag. Respective air content and durability factors were 4.5% and 90.6, 9% and 89.6, and 3.5% and 73.0. Again, it is clear that air content has a significant influence on durability; however, other factors can influence a concrete's overall durability. Mixture #7 (0.44/6.5/BFS50/II) with the lowest air content does in fact have the lowest durability, 3.5% and 73.0. Though this mixture contained the lowest air content, the durability factor was still greater than 60. Thus, the mixture is considered durable.

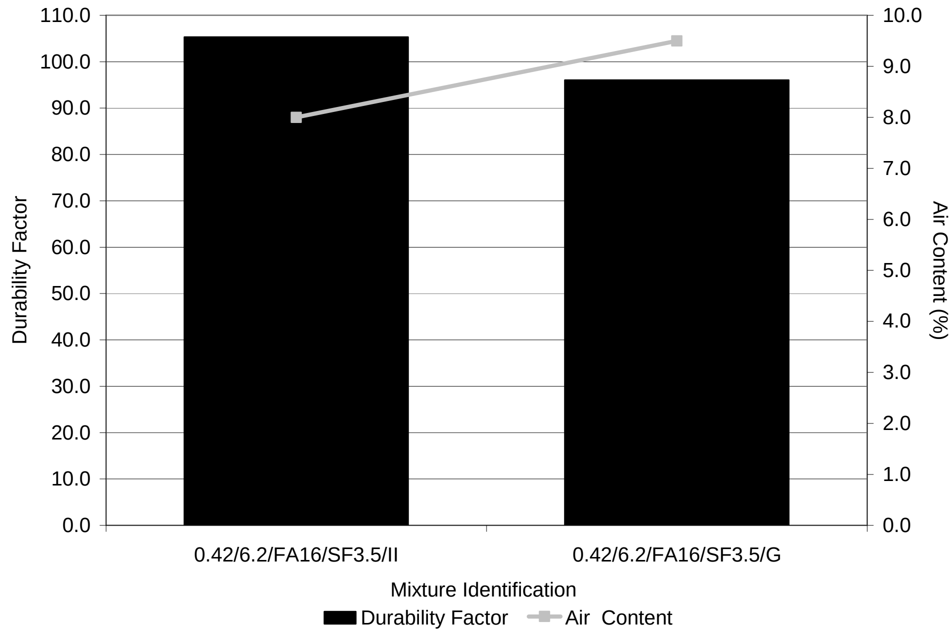

Figure 7.34 Durability Factor and Air Content, CDOT Control Mixture #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/G)

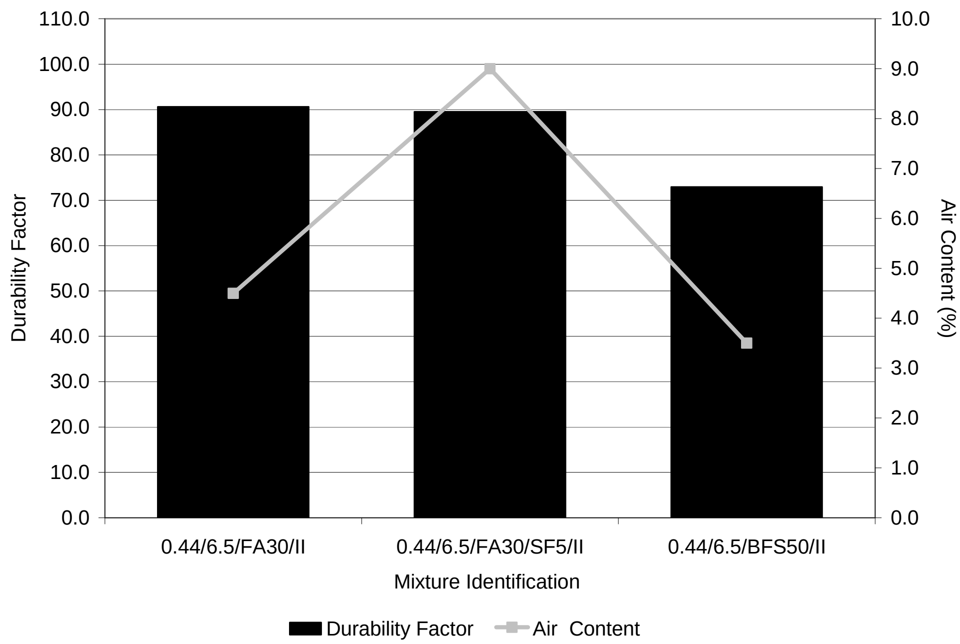

Figure 7.35 Durability Factor and Air Content, Mixture #5 (0.44/6.5/FA30/II), Mixture #6 (0.44/6.5/FA30/SF5/II), and Mixture #7 (0.44/6.5/BFS50/II)

7.3.3.2.3 Chemical Admixtures

Mixture #8 (0.44-6.0-FA30-SRA-II), made with a shrinkage reducing admixture, had the lowest air content (2.8%) of all eleven mixtures. Thus far, Mixture #8 is the only mixture whose relative modulus diminished below 60% before exposure to 300 freeze/thaw cycles. The test specimens for this mixture deteriorated much faster than those previously tested, having a relative modulus below 60% at only 60 freeze/thaw cycles. This is a direct result of low air content.

Mixtures #9 (0.44-6.0-FA30-RET-II) demonstrated superior durability having an average durability factor equal to 104.9. The mixture had an air content of 7.5% which helped the mixtures durability.

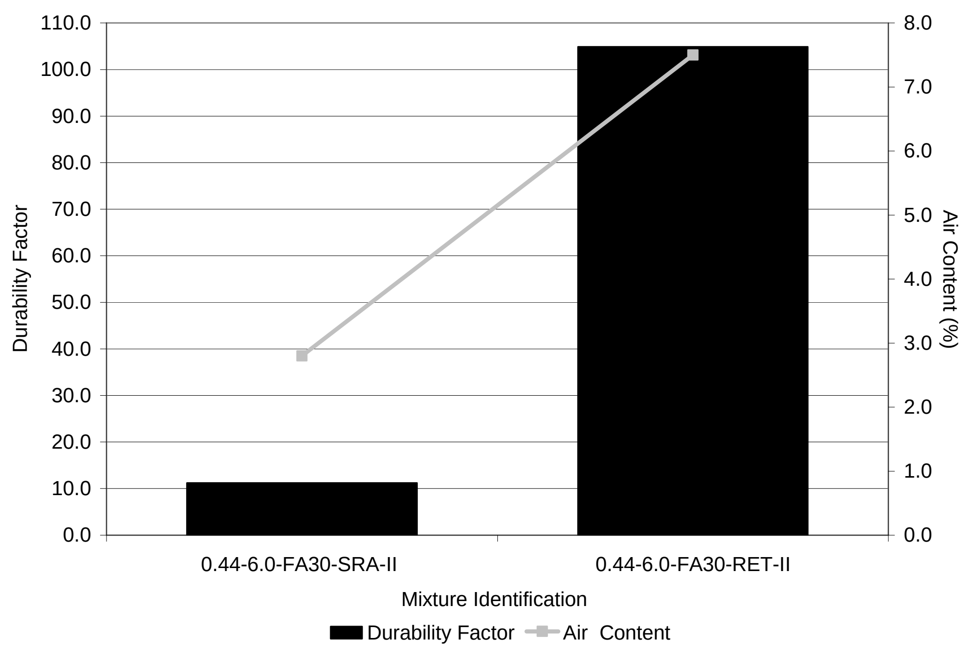

Figure 7.36 Durability Factor and Air Content, Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II)

7.3.3.2.4 Aggregate Type

Mixture #10 (0.42-6.0-II-Lightweight Aggregate) and Mixture #11 (0.42-6.0-II-Normal Weight Aggregate) have good durability. Both mixtures contained 7.5% air content. The normal weight mixture had a slightly lower durability factor than the lightweight mixture, 87.3 vs. 90.5. This is believed to be due to the continued hydration of the lightweight mixture beyond approximately 20 days of age.

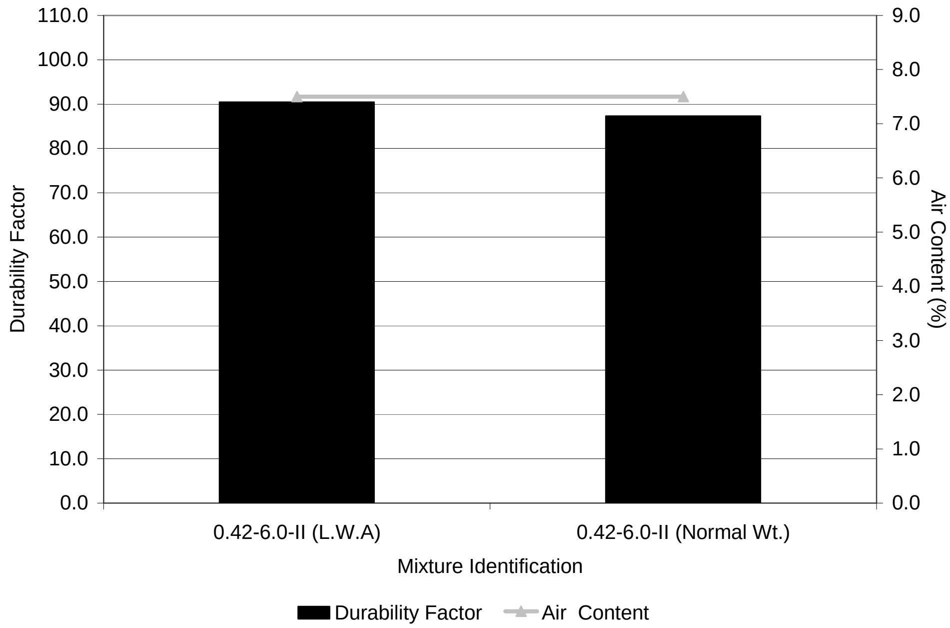

Figure 7.37 Durability Factor and Air Content, Mixture #10 (0.42-6.0-II-Lightweight Agg.), and Mixture #11 (0.42-6.0-II-Normal Weight Agg.)

7.3.4 Restrained Shrinkage Strain

7.3.4.1 General

The method used for this research to measure shrinkage was the restrained ring shrinkage test (ASTM C 1581, AASHTO PP34). Restrained shrinkage pertains to Class H and HT research because bridge decks are often cast in such a way to form a composite section with the girders below. In addition, the bridge decks are reinforced providing additional restraint. Over time the concrete undergoes volume change and attempts to shrink. The prevention of this shrinkage causes stresses, which translate into strain. Since bridge decks are suspended in the air, without earth for support or temperature absorption, they experience more shrinkage strain than the average reinforced roadway. Figure 7.38 is a photograph of a restrained ring shrinkage specimen.

Figure 7.38 Photograph of Restrained Ring Shrinkage Specimen (ASTM C 1581, AASHTO PP34)

Each steel ring was instrumented with four strain gauges which were mounted on the inside circumference, 90^{o} offset at mid-height. A more detailed description of the AASHTO PP34 (ASTM C 1581) test, including dimensions of the fabricated steel rings and procedures, is included in Appendix B. A program was written using the software configured for the data logger being used by the research team. The data logger is manufactured by Campbell Scientific. The program begins recording strain immediately and continues recording measurements in thirty-minute intervals. The program must be 'zeroed out' each time a new test is started. It requires one thirty minute interval to zero, another to take the first measurement, and one more before the measurements begin to stabilize and any external vibration removed. As a result, one or two of the initial strain measurements were sometimes omitted from the data because they were inconsistent. Two restrained shrinkage rings were fabricated for each mixture. The rings were immediately placed in a humidity controlled curing room (40% Relative Humidity) and at a temperature of 73 +/- 3^{o}F (23 +/- 2^{o}C). The dowels securing the concrete ring forms to the supporting form were removed and the rings were immediately covered and cured for 24 hours using wet burlap. The strain gauges connected to each ring were then connected to the data logger and the test initialized.

At 1 day of age, the outer mold of each ring was removed and any sharp corners (approximately 90^{o} top edges) were ground smooth and slightly round with a grinding stone. This was done to eliminate any accumulation of stresses at the edges (corners). Test durations per mixture varied depending upon whether or not the ring cracked and the rate of strain development. Four rings were continuously utilized allowing the research team to test two mixtures simultaneously. An additional steel ring instrumented with strain gauges was used to account for any temperature fluctuations. Figure 7.39 is a photograph of the AASHTO PP34 test setup.

Figure 7.39 Photograph of Restrained Ring Shrinkage Specimen (ASTM C 1581, AASHTO PP34)

Current CDOT Class H and HT specifications require concrete mixtures to not crack before 14 days of age. Tests were typically run for 28 to 30 days, and in some cases, over 50 days. The batching schedule for this research was primarily dictated by the designated amount of time needed for the cracking tendency test.

7.3.4.2 Strain Analysis

Concrete mixtures were compared on the basis of their individual strain development and magnitude at the time the test was discontinued. The rate of strength gain plays an important role in the rate of strain development. Discussion from previous sections analyzing compressive strength and development will be utilized in conjunction with the strain data for each mixture. Accelerated strength development results in a higher heat of hydration, or increased temperatures as cement hydrated during the initial set. Increased temperatures result in increased thermal stresses and increase the likelihood of cracking.

7.3.4.2.1 Cement Type

Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #3 (0.38-6.8-FA20-SF5-G) are identical mixtures but Mixture #3 used Type G, coarse ground cement. Coarse ground cement was incorporated into this research because it is believed to hydrate more slowly than normal Type II cement. The larger particles are expected to take longer to hydrate and develop strength at a slower rate. The reduced rate of strength gain should result in a lower heat of hydration and reduce thermal stresses. The reduced stresses should provide reduced strain and, in effect, a more crack resistant concrete. The restrained strain development for Mixtures #1 and #3 are shown graphically in Figure 7.40.

The Type G mixture did in fact develop strength more slowly than the Type II mixture; however it gained 26% more compressive strength by 56-days of age, 8712 vs. 6479psi, respectively. Mixture #3 (0.38-6.8-FA20-SF5-G) gained more ultimate strength than the Type II mixture; however, the rate of strength gain, particularly at early ages was reduced.

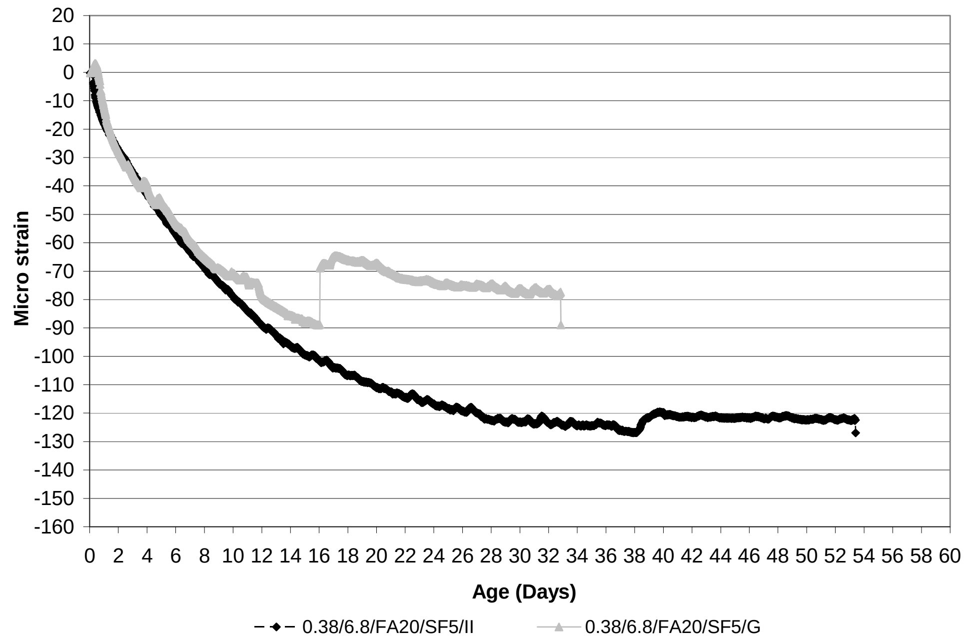

Figure 7.40 Restrained Shrinkage Strain, Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #3 (0.38-6.8-FA20-SF5- G) (ASTM C 1581, AASHTO PP34)

At 1, 3, and 7 days of age the Type II mixture and the Type G mixture gained the following respective percentages of their 56-day compressive strength; 32.9% vs. 15.7%, 59.9% vs. 44.5%, 71.5% vs. 60.5%. See Figure 7.41. The strain measurements do not follow the same trend. At the same days of age, the mixtures gained respective percentages of the ultimate strain of the concrete; 8.1% vs. -2.3%, 24.3% vs. 36.7%, 47.6% vs. 62.6%. Figure 7.42 shows the percentage of ultimate strain developed at 1, 3, 7, 28, and 56 days of age.

By 28-Days of age the coarse ground cement mixture had only achieved 85% of its ultimate strength while the Type II mixture had reached 96%. The Type II mixture proved to be more crack resistant than the Type G mixture. The test was terminated for Mixture #1 (0.38-6.8-FA20-SF5-II) at 54-days of age, while rings one and two were at an average of 122micro strain with no cracks. The mixture experienced a slight decrease in strain at 39 days but it was not a crack. Rings 1 and 2 for Mixture #3 (0.38-6.8-FA20-SF5-G) cracked at 16 days of age and an average of 90micro strain.

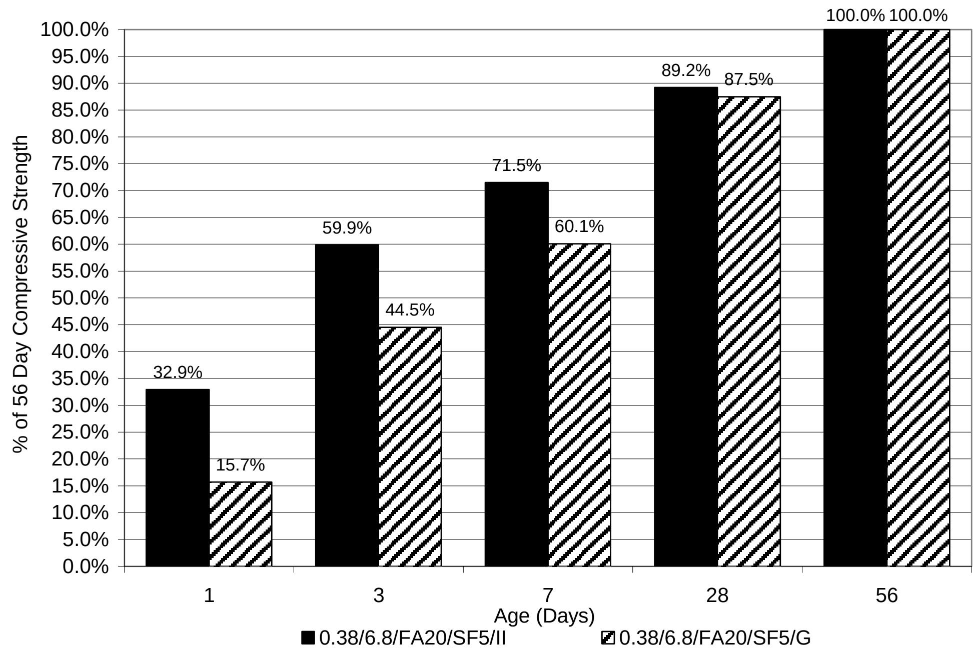

Figure 7.41 % of 56-Day Strength Achieved at Respective Age, Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #3 (0.38-6.8-FA20-SF5-G), (ASTM C 39. AASHTO T 22)

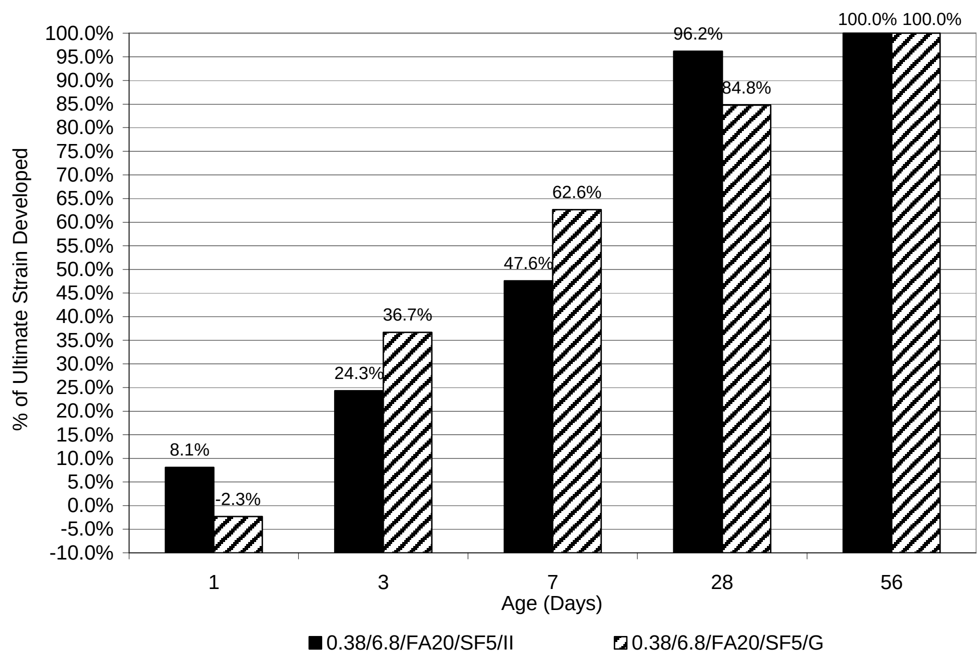

Figure7.42 % of Ultimate Strain Achieved, Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #3 (0.38-6.8-FA20-SF5-G), (ASTM C 1581, AASHTO PP34)

Mixture #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/ G) are identical mixtures but Mixture #4 is made using Type G, coarse ground cement, vs. Type II cement. A comparison of strain measurements for each mixture was expected to show reduced strain in concrete made with coarse ground cement. Figure 7.43 shows the rate of strength development of Mixtures #2 and #4.

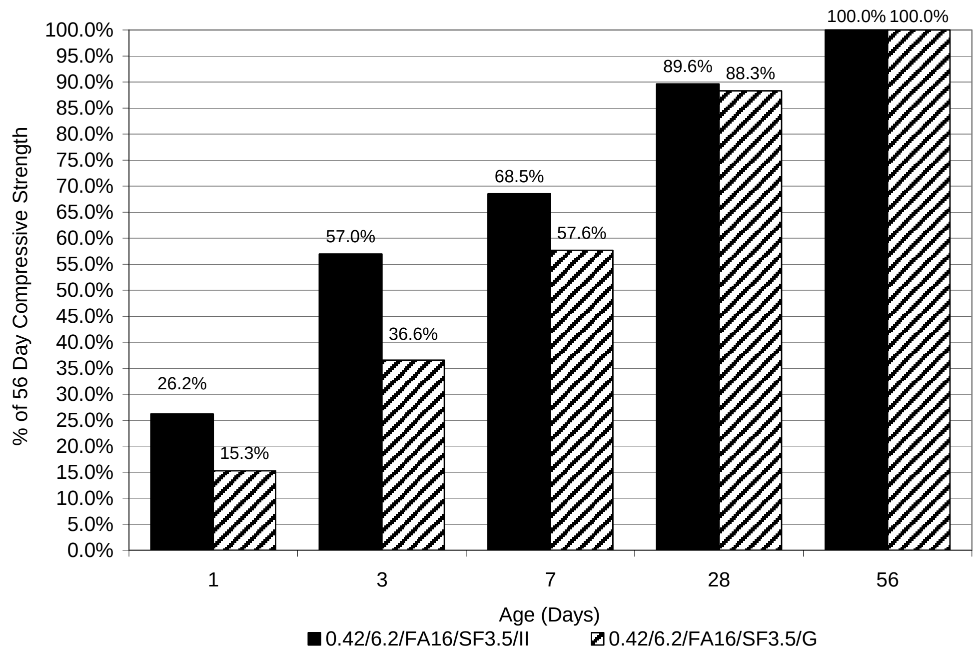

Figure 7.43 % of 56-Day Strength Achieved at Respective Age, Mixture #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/G), (ASTM C 39, AASHTO T 22)

Again, the coarse ground cement developed compressive strength at a lower rate than the Type II mixture. At 1 and 3 days of age, Mixture #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/G) developed 26.2% vs. 15.3% and 57% s. 36.6% of their 56 day compressive strength, respectively. The trend continues through 7 days of age when the mixtures achieved 68.5% vs. 57.6% of their ultimate strength, respectively. The Type G, coarse ground cement clearly reduces the rate of strength gain through 7 days of age.

A common trend with mixtures having w/cm of 0.42 and greater cementitious materials replacement percentage is a small number of negative strain measurements recorded in the beginning stages of the test. This is assumed to be a swelling of the concrete due to the excess water since it occurs less in mixtures having w/cm equal to 0.38 and more with a 0.44. As expected, the shrinkage strain developed according to the trend of strength development and the coarse ground cement mixture developed strain at a reduced rate. At 1, 3, and 7 days of age Mixtures #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/G) developed the following percentages of their ultimate strain; 10.1 vs. -3.9%, 28.5 vs. 25.8%, and 53.4 vs. 49.5%, respectively. The strain was only slightly reduced by the larger cement particles of the Type G mixture. At 28-Days of age the Type II mixture had achieved approximately100% of its ultimate strain because the strain had leveled off before cracking at 29 days of age at an average of 108 micro strain. Although the coarse ground cement mixture developed strain at a reduced rate, the mixture cracked at a smaller magnitude of strain than the Type II mixture (95 vs. 108micro strain) at 24 days of age. Figure 7.44 provides the percentage of strain development for Mixtures #2 and #4.

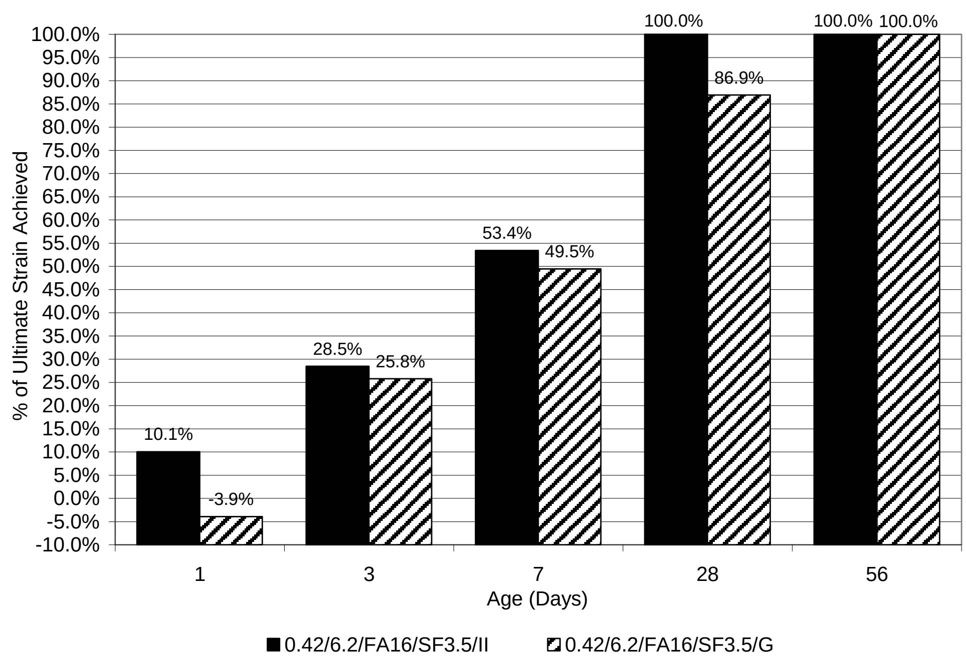

Figure 7.44 % of Ultimate Strain Achieved at Respective Age, Mixture #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/G), (ASTM C 1581, AASHTO PP34)

The shrinkage strain versus concrete age is plotted in Figure 7.45

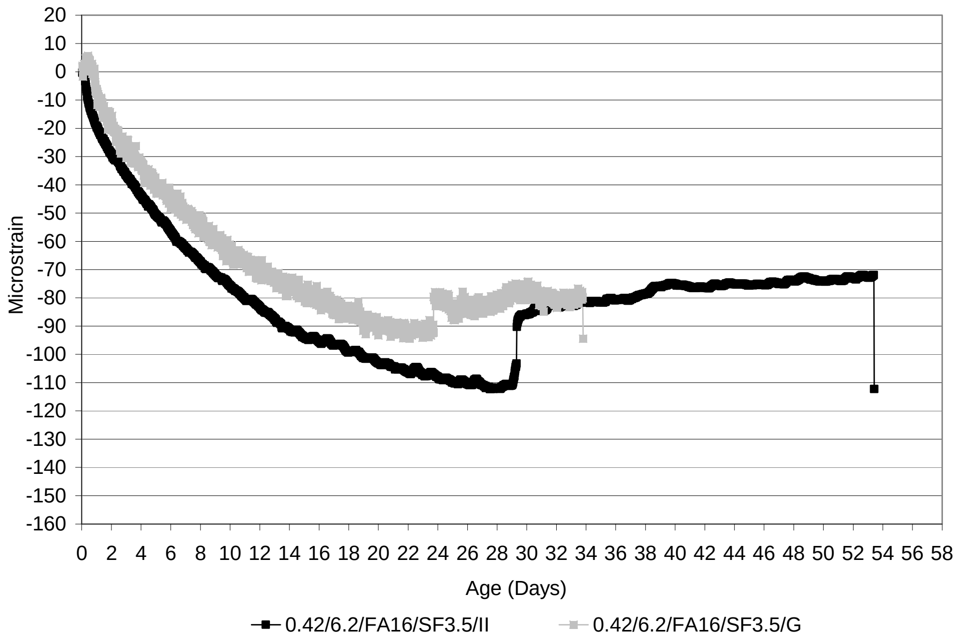

Figure 7.45 Restrained Shrinkage Strain, CDOT Control Mixture #2 (0.42/6.2/FA16/SF3.5/II) and Mixture #4 (0.42/6.2/FA16/SF3.5/G), (ASTM C 1581, AASHTO PP34)

As with the previous CDOT Class H control mixture comparison, the Type G, coarse cement mixture did not prove to be beneficial in producing a more crack resistant concrete. Both CDOT Class H control mixtures, Mixture #1 (0.38-6.8-FA20-SF5-II) and Mixture #2 (0.42/6.2/FA16/SF3.5/II), proved to be effective against shrinkage strain during the restrained ring shrinkage test (ASTM C 1581, AASHTO PP34).

7.3.4.2.2 Supplementary Cementitious Materials

Mixtures #5 (0.44/6.5/FA30/II), #6 (0.44/6.5/FA30/SF5/II), and #7 (0.44/6.5/BFS50/II) all have the same w/cm (0.44) but each introduces various amounts of cement replacement with other supplementary cementitious materials; 30% Class F fly ash, 30% Class F fly ash with 5% silica fume, and 50% blast furnace slag.

All three mixtures have higher w/cm and replacement percentages of Class F fly ash than is currently allowable per CDOT Class H and HT specifications. In addition, the incorporation of 50% replacement of cement with ground-granulated blast furnace slag in Mixture #7 is not allowable per current CDOT specifications. All three mixtures developed shrinkage strain at a very slow rate, measuring 1 day strains still negative; -7.0, -4.7, and -4.3micro strain, respectively. Negative strain values are common among the initial strain measurements previously recorded when the test is initialized. Mixtures previously tested with w/cm equal or greater than 0.42 have demonstrated this trend. Mixture #5 (0.44/6.5/FA30/II) and #6 (0.44/6.5/FA30/SF5/II) both contain an increased (30%) replacement of cement with Class F fly ash. In addition to the fly ash, Mixture #6 introduces a 5% replacement of cement with silica fume. This is the highest allowable replacement of cement with silica fume per current CDOT Class H and HT specification. Figure 7.46 provides the shrinkage strain for Mixtures #5, #6, and #7.

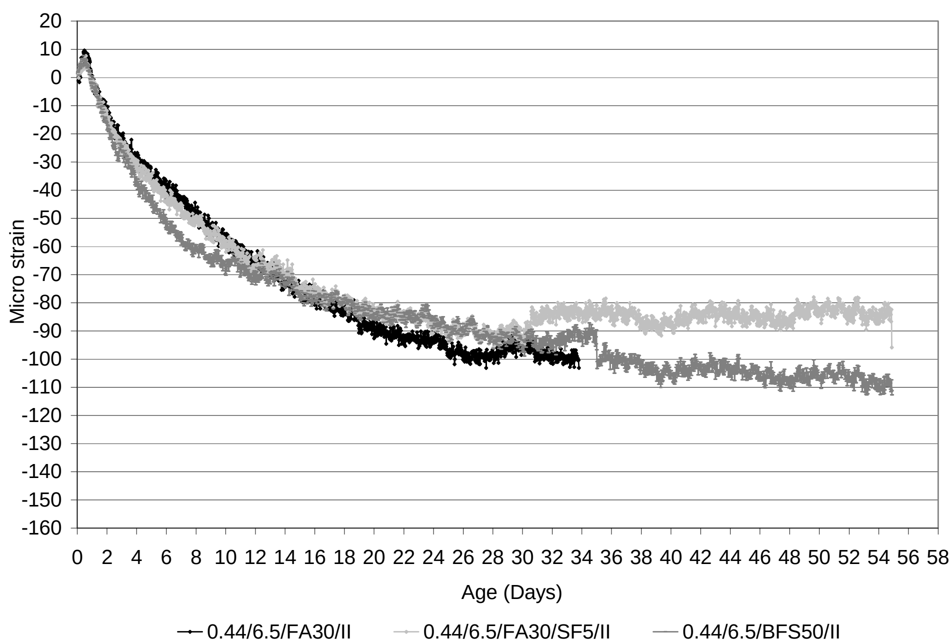

Figure 7.46 Restrained Shrinkage Strain, Mixture #5 (0.44/6.5/FA30/II), Mixture #6 (0.44/6.5/FA30/SF5/II), and Mixture #7 (0.44/6.5/BFS50/II), (ASTM C 1581, AASHTO PP34)

The development of strength gain was similar between Mixture #5 (0.44/6.5/FA30/II) and #6 (0.44/6.5/FA30/SF5/II). The air content of the silica fume mixture decreased the magnitude of strength but had a negligible affect on the rate of strength or strain development. At 3 days of age, both mixtures developed approximately 20% of their ultimate strain. At 7 days of age the silica fume mixture gained approximately 48% of its ultimate strain and the fly ash only mixture gained approximately 40%. See Figure 7.47.

By 28-days of age the mixture made using only fly ash replacement gained approximately the same amount of its ultimate strain as the fly ash and silica fume mixture, 94.8 and 94.2%, respectively. The silica fume reduced the magnitude of the ultimate strain for Mixture #6 (0.44/6.5/FA30/SF5/II) by 7micro strain, 96 vs. 103micro strain. Mixture #7 (0.44/6.5/BFS50/II) developed shrinkage strain at similar rates to Mixture #5 (0.44/6.5/FA30/II) and #6 (0.44/6.5/FA30/SF5/II) at 1, 3, and 7 days of age. The blast furnace slag mixture produced a higher magnitude of shrinkage strain than the mixtures made using only fly ash replacement and with the addition of silica fume; 113micro strain vs. 103 and 96micro strain, respectively. Mixture #7 (0.44/6.5/BFS50/II) achieved a higher ultimate strength than Mixture #5 (0.44/6.5/FA30/II) and #6 (0.44/6.5/FA30/SF5/II); 6976psi vs. 5467 and 4298psi, respectively.

Mixture #5 (0.44/6.5/FA30/II) cracked at 28 days at an average of approximately 100micro strain. Mixture #6 (0.44/6.5/FA30/SF5/II) cracked at 31 days with an average of approximately 95micro strain. Mixture #7 (0.44/6.5/BFS50/II) cracked at 32 days at an average of approximately 90micro strain, although the strain continued to gradually increase to an ultimate strain of 113micro strain when the test was discontinued.

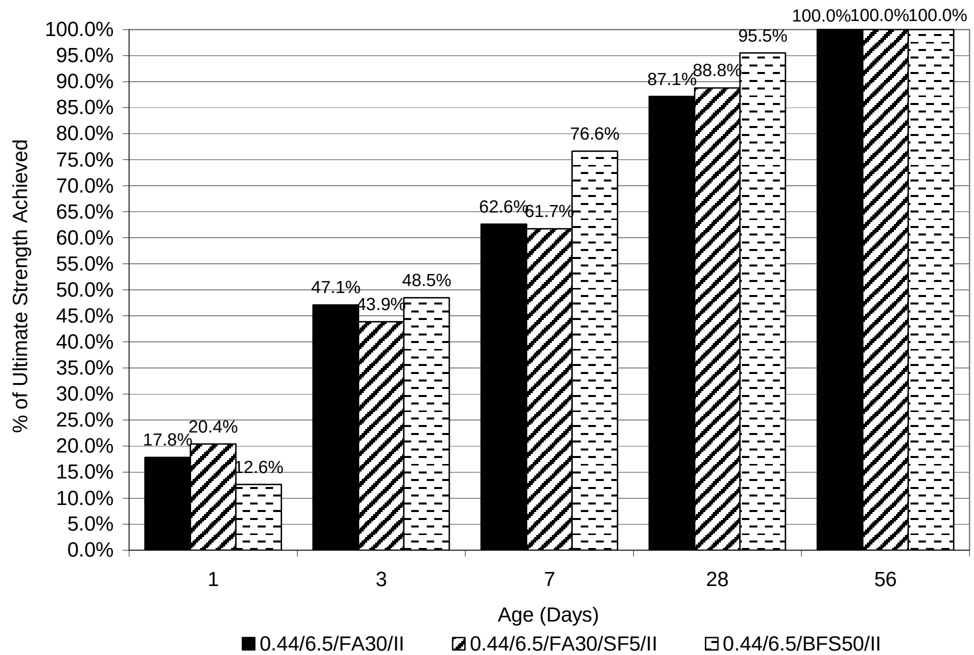

Figure 7.47 % of Ultimate Strain Achieved at Respective Age, Mixture #5 (0.44/6.5/FA30/II), Mixture #6 (0.44/6.5/FA30/SF5/II), and Mixture #7 (0.44/6.5/BFS50/II), (ASTM C 1581, AASHTO PP34)

7.3.4.2.3 Chemical Admixtures

A comparison of strain measurements was performed on the two mixtures incorporating chemical admixtures. Mixture #8 (0.44-6.0-FA30-SRA-II) incorporated a shrinkage reducing admixture, Master Builders- Tetraguard_AS20, and Mixture #9 (0.44-6.0-FA30-RET-II) used a set retarder, Master Builders- Pozzolith_100XR.

The maximum dosage rate of the SRA was used in Mixture #8 (0.44-6.0-FA30-SRA-II), at 1.5gal./yd.^{3}, or 0.19 gallons per the 3.5ft.^{3} batch size. This converted to 736.1mL per batch. Cost benefit analysis was not included in the scope of this research, but at the maximum dosage rate it is easy to see how the use of such admixtures could quickly increase a large concrete-project budget. The average dosage rate of the set retarder was used in Mixture #9 (0.44-6.0-FA30-RET-II), at 3 ounces per one hundred pounds of cementitious materials. For the batch having 540lbs/yd.^{3} of combined cement and fly ash, 16oz./yd.^{3} or, 473.1mL/yd.^{3}, of the retarder was used in Mixture #9.

Restrained ring shrinkage test (ASTM C1581, AASHTO PP34) results for Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II) are plotted in Figure 7.48.

The shrinkage reducing admixture proved to be very effective against shrinkage strain, achieving an ultimate strain of only 73micro strain at 56-days of age. This was the smallest magnitude of strain achieved by any of the mixtures in the first two to three weeks of testing, and exceptionally at 56-days of age. While the development of strain was decreased significantly, strength development was normal and not reduced, as it achieved approximately 75% of its 28 day strength at 7 days of age, 3496 of 4817psi, respectively. Figure 7.49 shows the strength development through 56 days of age for Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II).

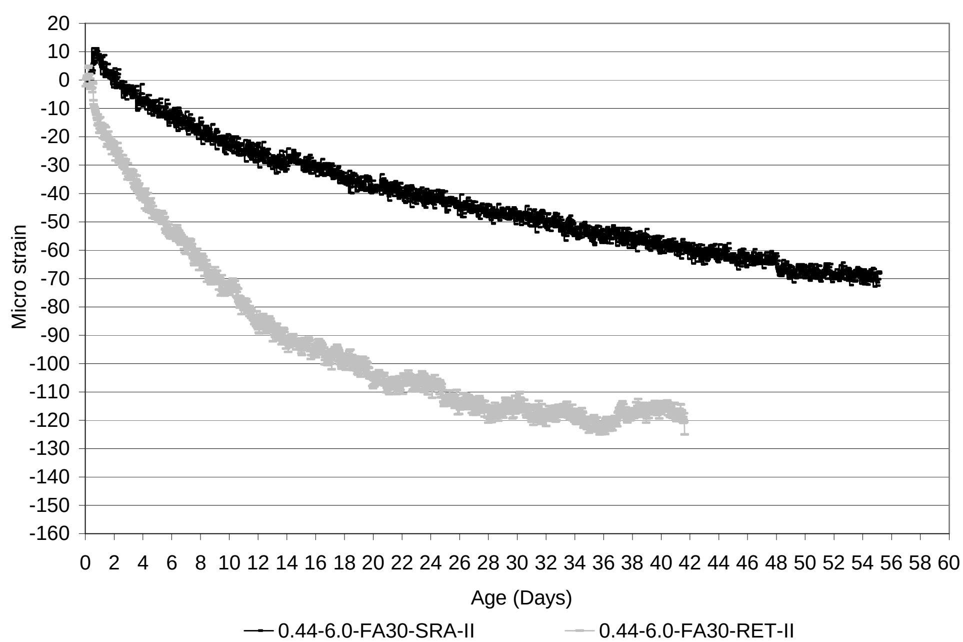

Figure 7.48 Restrained Shrinkage Strain, Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II), (ASTM C 1581, AASHTO PP34)

The early age compressive strength of the SRA mixture developed at a comparable rate to the set retarder mixture. At 7 days of age, Mixture #8 (0.44-6.0-FA30-SRA-II) experienced slightly less compressive strength; however, has similar strength at 28 and

56 days of age. At 28 days of age, Mixture #8 (0.44-6.0-FA30-SRA-II) achieves similar compressive strength to Mixture #9 (0.44-6.0-FA30-RET-II), within 1%, at 4817 vs. 4806psi, respectively. The mixtures achieved 56 day compressive strengths within 2% of one another, 5572 vs. 5685psi respectively, and at the same time the ultimate strain of the SRA mixture was reduced by 42% from the set retarder mixture, 125 vs. 73micro strain respectively. Mixture #9 (0.44-6.0-FA30-RET-II) only reduced the development of early age strain slightly at 1 day of age, before accelerating past the SRA mixture at 7 days of age. The ultimate strength was not affected but the development of strain was greatly increased. .

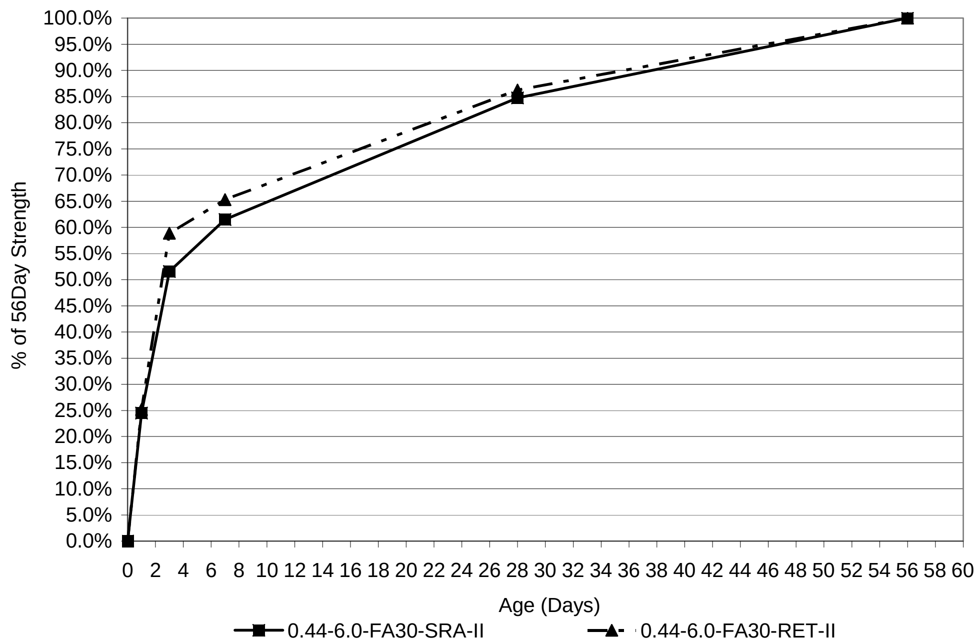

Figure 7.49 % of 56-Day Strength Achieved, Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II), (ASTM C 39, AASHTO T 22)

The test was discontinued at 57 days of age due to time constraints on this research study. At 56-days of age, Mixture #8 (0.44-6.0-FA30-SRA-II) shrinkage rings experienced considerably less shrinkage strain than all other mixtures examined in this study. In addition, the Mixture #8 rings did not exhibit a crack prior to termination. Mixture #9 (0.44-6.0-FA30-RET-II) cracked at 36 days of age and approximately 128micro strain. The shrinkage reducing admixture proved to be very effective when used at the maximum dosage rate. Development of strength was adequate and shrinkage strain was greatly reduced as a result of the admixture. The air content for the mixture was only 2.8% due to the SRA interaction and, as a result, the mixture exhibited poor freeze/thaw durability.

At 7 days of age, the set retarder mixture achieved 43% of its ultimate strain while the SRA. mixture reached 21%, 53 vs. 15micro strain respectively. The trend continued at 28 days of age as the set retarder mixture achieved 92% of its ultimate strain vs. the SRA reaching only 62%, 115 vs. 45micro strain. The set retarder strain measurements are not exceptionally high in magnitude of micro strain but the rate at which the mixture developed the strain is quite high. Increased development rates of strain often lead to cracking in the field and are not beneficial. The percent of ultimate strain is shown in Figure 7.50.

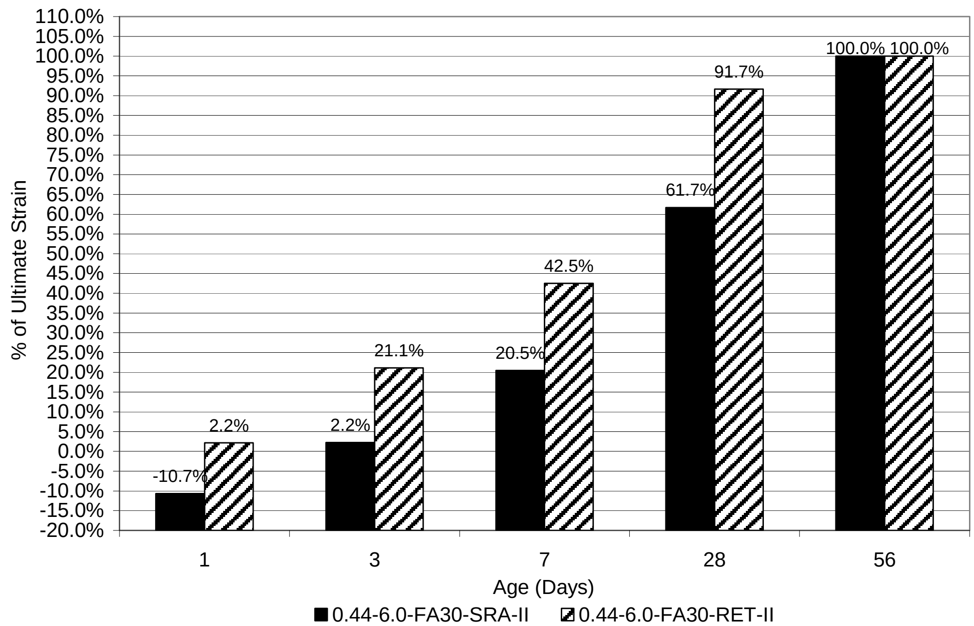

Figure 7.50 % of Ultimate Strain Achieved, Mixture #8 (0.44-6.0-FA30-SRA-II) and Mixture #9 (0.44-6.0-FA30-RET-II), (ASTM C 1581, AASHTO PP34)

7.3.4.2.4 Aggregate Type

Mixture #10 (0.42-6.0-II-L.W.A) and Mixture #11 (0.42-6.0-II-Norm.Wt.) are identical; however, Mixture #10 substituted 250lbs./yd.^{3} of the fine aggregate with lightweight fine aggregate. The aggregate had been pre-conditioned (pre-soaked) to a moisture content (MC) of approximately 18%. This is an exceptionally high MC. for any aggregate but is done so with the intent of internally curing the concrete. Over time, the aggregate releases internal water that promotes continued hydration of the cement particles. Figure 7.51 shows the development of strength through 56 days of age.

Strength development is only slightly decreased beyond 1 day of age. The lightweight aggregate is made of expanded shale and is weaker in shear than normal limestone or quartz aggregate. Results show 28 day compressive strengths to be comparable as increased hydration past 7 days of age causes the rate of strength gain to recover to within 2% of the normal weight aggregate mixture, 5678 vs. 5807psi for Mixtures #10 and #11 respectively. By 56 days of age the continued internal curing from the lightweight aggregate (LWA) mixture developed 6% more compressive strength than the normal weight aggregate (NWA)mixture; 6273 vs. 5879psi.

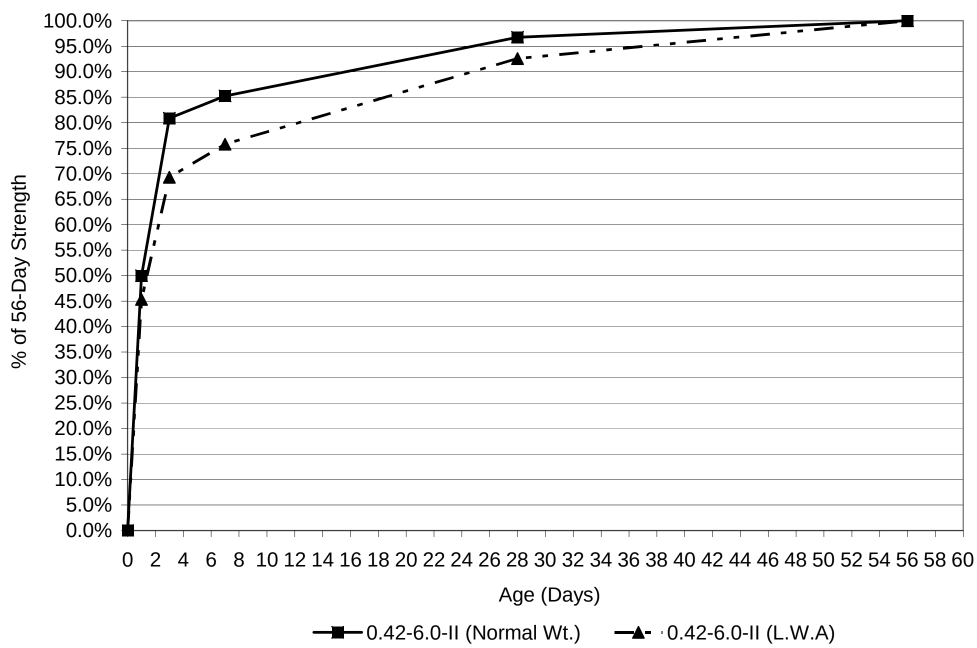

Figure 7.51 % of 56-Day Strength Achieved, Mixture #10 (0.42-6.0-II-L.W.A) and Mixture #11 (0.42-6.0-II-Norm.Wt.), (ASTM C 39, AASHTO T 22)

The LWA mixture developed strain at a decreased rate than the NWA mixture. The LWA and NWA mixtures reached 21% vs. 29% of their ultimate strain at 3 days of age respectively. By 7 days of age, the LWA mixture reached 47% of its ultimate strain and the NWA mixture 57%. By 28 days of age Mixture #10 (0.42-6.0-II-L.W.A) achieved

8% less shrinkage strain than Mixture #11 (0.42-6.0-II-Norm.Wt.); 110 s. 119micro strain respectively. The LWA mixture achieved an ultimate strain equal to 125micro strain at 32 days while the NWA mixture achieved 134micro strain at 34 days. The percent of ultimate strain is shown in Figure 7.52.

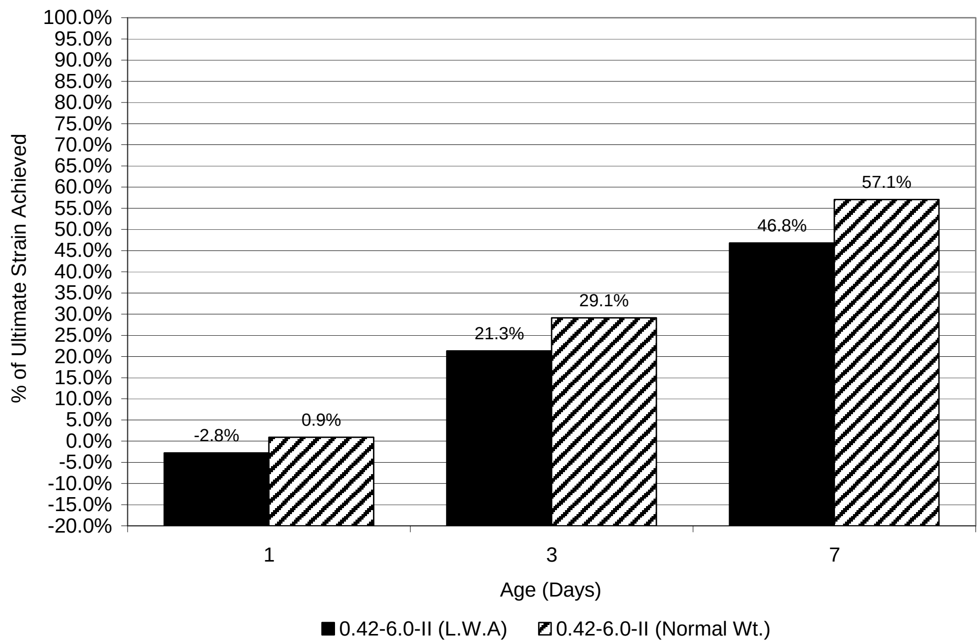

Figure 7.52 % of Ultimate Strain Achieved, Mixture #10 (0.42-6.0-II-L.W.A) and Mixture #11 (0.42-6.0-II-Norm.Wt.), (ASTM C 1581, AASHTO PP34)

Mixture #10 (0.42-6.0-II-L.W.A) shrinkage rings cracked at 32 days and an average of approximately 125micro strain. Mixture #11 (0.42-6.0-II-Norm.Wt.) shrinkage rings cracked at 34 days and an average of approximately 134micro strain. The lightweight aggregate mixture developed shrinkage strain at a reduced rate to the normal weight mixture from 3 days of age onward. The LWA mixture cracked only two days prior to the normal weight mixture, 32 vs. 34 days, and at a magnitude of shrinkage strain of only 7% less. The use of lightweight aggregate in Mixture #10 proved to be helpful in reducing restrained shrinkage strain development. However, the LWA restrained shrinkage ring cracked at a lower magnitude of strain. Future evaluations may include increased percentages of LWA. The shrinkage strain for Mixtures #10 and #11 are shown in Figure 7.53.

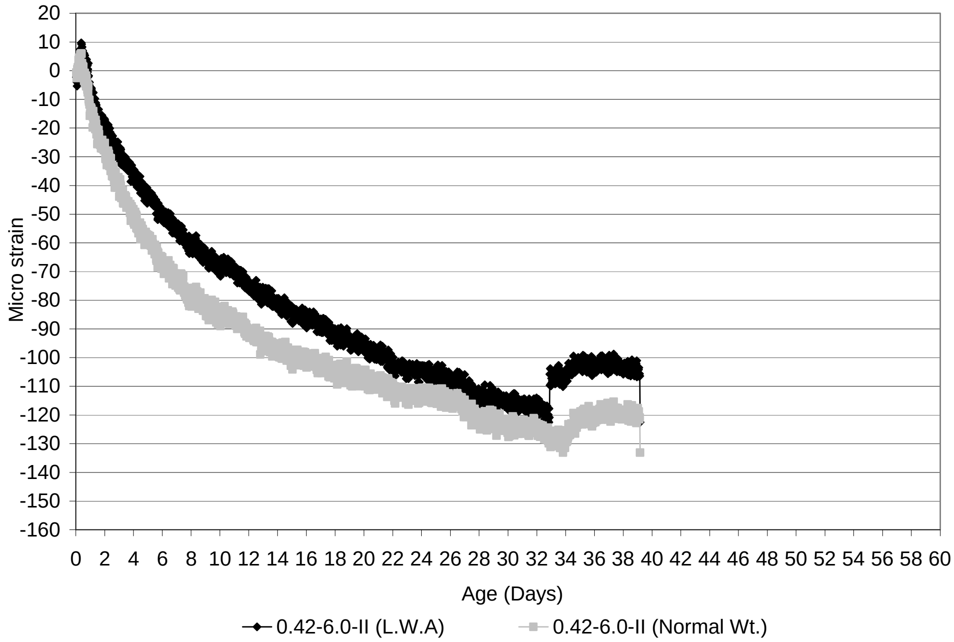

Figure 7.53 Restrained Shrinkage Strain, Mixture #10 (0.42-6.0-II-L.W.A) and Mixture #11 (0.42-6.0-II-Norm.Wt.), (ASTM C 1581, AASHTO PP34)

7.3.4.3 Paste Content (Volume)

Shrinkage is a paste property of concrete. Thus, the volume shrinkage that occurs is the paste shrinking and not the fine or coarse aggregate. The aggregate restrains against this shrinkage and causes the concrete to crack. One approach is to minimize the paste content (%) in a concrete mixture and therefore, less paste should equate to less shrinkage. A comparison of paste content and the development of strain is made for the eleven mixtures designed and tested in this study. Table 7.29 lists each of the mixture properties including paste content.

Table 7.29 Mixture Design Characteristics

Mixture ID	w/cm	Cementitious Content	Type of Cement	Admixture	Air Content (%)	Paste Volume
0.38/6.8/FA20/SF5/II	0.38	640	Type II		6.5	28%
0.42/6.2/FA16/SF3.5/II	0.42	580	Type II		6.5	26%
0.38/6.8/FA20/SF5/G	0.38	640	Class G Oil Well Cement (Coarse Grained Cement)		6.5	28%
0.42/6.2/FA16/SF3.5/G	0.42	580	Class G Oil Well Cement (Coarse Grained Cement)		6.5	26%
0.44/6.5/FA30/II	0.44	611	Type II		6.5	29%
0.44/6.5/FA30/SF5/II	0.44	611	Type II		6.5	29%
0.44/6.5/BFS50/II	0.44	611	Type II		6.5	28%
0.44/6.0/FA30/SRA/II	0.44	540	Type II	SRA	6.5	25%
0.44/6.0/FA30/RET/II	0.44	540	Type II	RET	6.5	25%
0.42/6.0/II-L.W.A.	0.42	564	Type II		6.5	25%
0.42/6.0/II-Norm.Wt.	0.42	564	Type II		6.5	25%

Paste content has long been recognized as a factor in concrete shrinkage. Moderate paste content was a priority in the designing of concrete mixtures used for this research. An average paste content of 28% was consistent for several of the mixtures. For the benefit of this research some of the mixtures were designed with paste contents slightly higher than what is ideal. This was done to examine the effect those cementitious materials had on shrinkage. The excess paste provided a clearer result of exactly how these cementitious materials effect shrinkage.

The two mixtures having the highest paste content are Mixtures # 5 (0.44/6.5/FA30/II) and #6 (0.44/6.5/FA30/SF5/II). The two mixtures reached average ultimate strains that are comparable with the other mixtures, 103 and 93micro strain. The two mixtures having the lowest paste content (25%), Mixtures #8 and #9, reached an ultimate strain of (73 vs. 105micro strain). Both are comparable to the 29% paste content mixtures. It should be noted that all four mixtures have w/cm equal to 0.44.

At 1, 3, 7, and 28 days of age, Mixture #8 (0.44/6.0/FA30/SRA/II) with 25% paste content achieved less of its ultimate strain at respective days than any of the other mixtures. However, this is not an accurate representation of 25% paste content. Mixture

#8 incorporated a shrinkage reducing admixture which decreased its ultimate strain as well as its strain development at all ages.

At 1 day of age the 25% paste content mixture containing set retarder achieved more of its ultimate strain than both of the 29% paste content mixtures. From 3 to 7 days of age Mixture #5 (0.44/6.5/FA30/II), Mixture #6 (0.44/6.5/FA30/SF5/II), and Mixture #9 (0.44/6.0/FA30/RET/II) reached comparable percentages of their ultimate strain; 19.4, 21.5, and 21.1% at 3 days, respectively. Mixture #5 has 29% paste content and developed strain at a comparable, but increased rate when compared to 25% paste content mixtures.

By 28 days of age, increased paste content only slightly increased shrinkage strain. Mixture 5 (0.44/6.5/FA30/II) and Mixture #6 (0.44/6.5/FA30/SF5/II) having 29% paste content reached 95 and 94% of their ultimate strain, while Mixture #9 (0.44/6.0/FA30/RET/II) was similar with 92%. At early ages, increased paste content of 4% only slightly increased development of strain. In fact, the ultimate strain of Mixture #9 (0.44/6.0/FA30/RET/II) surpassed the 25% paste content mixtures by approximately 10 to 15micro strain. Figure 7.54 illustrates the effects of past content on percent of ultimate strain.

A mixture having increased w/cm and fly ash replacement but having a 5% addition of silica fume decreased both the ultimate and rate of development of strain. It is possible that Mixture #9 (0.44/6.0/FA30/RET/II) achieved the highest strain due to its more rapid increase in strength gain after initial set.

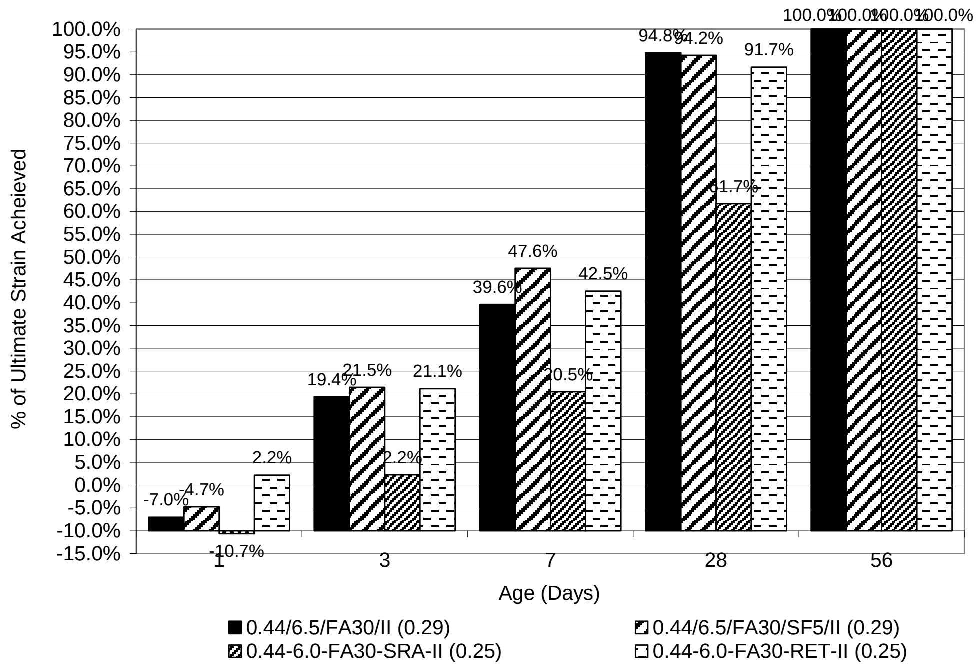

Figure 7.54 % of Ultimate Strain Achieved vs. Paste Content (29 vs. 25%), Mixture 5 (0.44/6.5/FA30/II) and Mixture #6 (0.44/6.5/FA30/SF5/II) vs. Mixture #8 (0.44/6.0/FA30/SRA/II) and Mixture #9 (0.44/6.0/FA30/RET/II) (ASTM C 1581, AASHTO PP34)

Mixture #1 (0.38/6.8/FA20/SF5/II) and Mixture #2 (0.42/6.2/FA16/SF3.5/II) are CDOT Class H and HT control mixtures having paste contents of 28 and 26% respectively. Mixture #2 has an increased w/cm but less cement than Mixture #1, resulting in the decreased 2% paste content. At 1 day of age both mixtures have low strain values but each reach approximately 10% of their ultimate strain. By 3 and 7 days of age, Mixture #2 (0.42/6.2/FA16/SF3.5/II) with an increased w/cm (0.42 vs. 0.38) and decreased paste content (26 vs. 28%) achieved 15% and 11% more of its ultimate strain than Mixture #1 (0.38/6.8/FA20/SF5/II), respectively.

By 28 days of age, Mixture #2 (0.42/6.2/FA16/SF3.5/II) with 26% paste content had achieved 100% of its ultimate strength while Mixture #1 (0.38/6.8/FA20/SF5/II) with 28% paste content only reached 96%. Mixture #2 (0.42/6.2/FA16/SF3.5/II) achieved higher ultimate strain with 26% paste content than Mixture #1 (0.38/6.8/FA20/SF5/II) with 28% paste content. The increased w/cm is believed to be the reason for the increased strain of 12% (127 vs. 112micro strain respectively). See Figure 7.55.

Paste content didn't seem to affect shrinkage strain alone. Mixtures with increased w/cm and less paste content achieved higher ultimate strains than those with increased paste content and decreased w/cm.

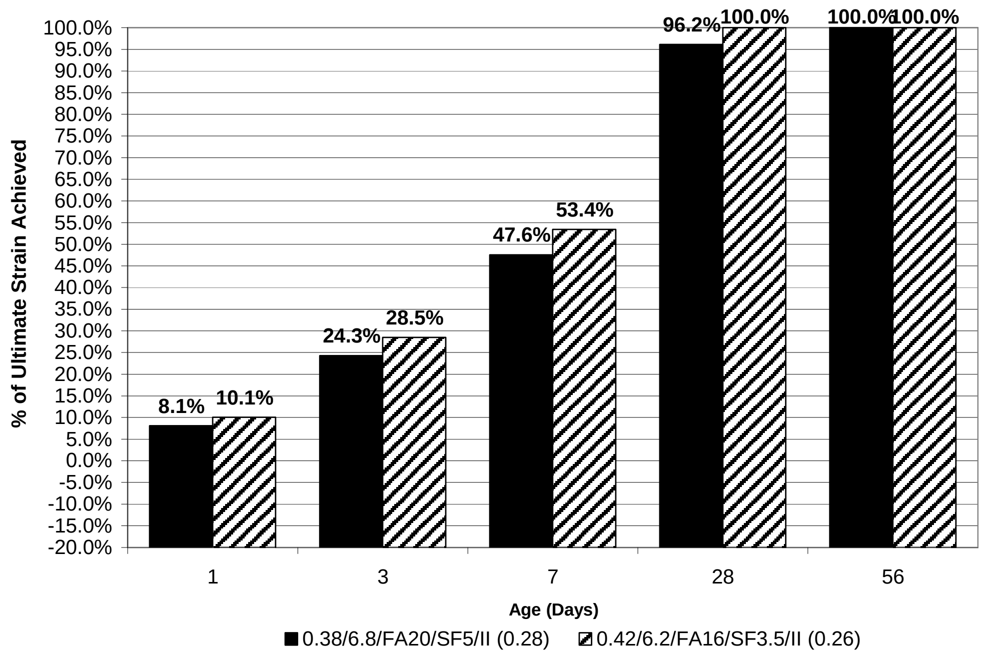

Figure 7.55 % Ultimate Strain Achieved vs. Paste Content (28 vs. 26%), Mixture #1 (0.38/6.8/FA20/SF5/II) and Mixture #2 (0.42/6.2/FA16/SF3.5/II) respectively, (ASTM C 1581, AASHTO PP34)

CHAPTER 8 - CONCLUSIONS AND RECOMMENDATIONS

This report evaluated the current CDOT Class H and HT concrete mixture specification. In addition, nine other mixtures were investigated to aid in the development of a more crack resistant concrete specification. In total, eleven concrete mixtures were design, batched, and tested for their fresh and hardened concrete performance. Specifically, the designs differed by type of cement, w/cm, cement content, SCMs,, use of chemical admixtures, and aggregate type. Compressive strength, permeability, freeze-thaw resistance, and restrained shrinkage cracking were evaluated and reported in this report. A summary of the major findings from this study are reported below.

8.1 Fresh Concrete Properties

8.1.1 Slump

Slump values were increased slightly with the use of Type G, coarse-ground cement. In addition, an increase in slump was also observed when the percentage of cement replacement with fly ash was increased beyond the current replacement levels.

8.1.2 Air Content

The air content varied between mixtures. The Type G cement didn't seem to have any affect on air content at a w/cm of 0.38. However, at w/cm of 0.42 the air content greatly increased with the Type G cement concrete mixture.

The use of chemical admixtures greatly reduced air content. Shrinkage reducing admixtures reduced the air content within the concrete significantly. Increased percent cement replacement with SCMs increased the workability of the mixtures. When necessary, careful addition of HRWRA extended mixing times. Increased time in the mixer deflates the mixture and results in a decreased air content. The set retarder increased air content slightly with only an average recommended dosage rate.

8.1.3 Unit Weight

Unit weight for all eleven mixtures varied due to the fluctuation in air content. When the design "predicted" unit weight was adjusted for the measured "actual" air content, the revised unit weight was reasonably close to the measured value. The LWA mixture did not produce a lightweight concrete. It produced a concrete of comparable unit weight to the other eleven mixtures. Some of the mixtures with the highest w/cm resulted in the largest unit weight. This is again due to low air content.

8.1.4 Temperature

Ambient and concrete temperatures were average and within appropriate ranges for concrete placement. Temperature is not believed to have played a significant role in this study.

8.2 Mixture Design Properties

8.2.1 General

Lower w/cm will result in high early compressive strengths and rates of strength and strain development. Increasing the w/cm to 0.44 and Class F fly ash replacement levels up to 30% was beneficial in controlling strength gain. Mixture 5 (0.44/6.5/FA30/II) did so, resulting in a comparable rate of strength development to its control mixture but decreased the strain development and 56-day ultimate strength. A low cement content mixture with increased w/cm and fly ash replacement proved to be beneficial. When SCMs are not utilized, a low cement content of 6.0 bags is beneficial. When SCMs are used, increased cement content may be necessary to maintain the same properties.

Type G, coarse-ground cement was beneficial to strain and strength at the higher w/cm of 0.42 and low cementitious materials content. At lower w/cm of 0.38 the cement behaved similarly to the control mixture fabricated using Type II cement, developing strain and strength at an average rate.

A high dosage rate of a shrinkage reducing admixture is extremely beneficial in controlling both the development rate and ultimate strain of the mixture, while maintaining adequate development of ultimate strength at all ages. An average dosage rate of a set retarder only retarded the initial strength development slightly. After 1 day of age, the development of strength and strain was substantially increased. Although the concrete containing the set retarder reached higher compressive strengths more quickly than anticipated, the concrete did not crack in the AASHTO PP34 test and was moderately durable.

Table 8.1 shows the 56 days of age compressive strength and permeability results. In addition, the results of the restrained shrinkage test are included. The mixture designs batched were used as a basis for analysis and variations and are utilized in developing recommendations to current Class H and HT specifications. Table 8.2 compares the mixture designs examined in this study with the Class H and HT specification requirements for compressive strength, permeability, and cracking tendency.

Table 8.1 Compressive Strength, Permeability, and Restrained Shrinkage Test Results

Mixture Number	Mixture Identification	56-day f'c (psi)	56-day Permeability (Coulombs)	Crack at 14-Days	Ring Status & Age at Max. Strain (Days)	Maximum Strain (microstrain)
1	0.38/6.8/FA20/SF5/II	6479	596	No	Ring 1_39 days	-127
2	0.42/6.2/FA16/SF3.5/II	4643	835	No	Ring2_29days	-112
3	0.38/6.8/FA20/SF5/G	8712	373	No	Ring1_16.5 days	-89
4	0.42/6.2/FA16/SF3.5/G	3931	1965	No	Ring1_24days	-94
5	0.44/6.5/FA30/II	5467	1789	No	Rings Did Not Crack	-103
6	0.44/6.5/FA30/SF5/II	4298	1387	No	Ring 1_31days	-96
7	0.44/6.5/BFS50/II	6976	991	No	Ring 1_32days	-113
8	0.44-6.0-FA30-SRA-II	5685	1400	No	Rings Did Not Crack	-73
9	0.44-6.0-FA30-RET-II	5572	1622	No	Rings 1&2_36 days	-125
10	0.42-6.0-II (L.W.A)	6273	1529	No	Ring 1_32.5days	-125
11	0.42-6.0-II (Normal Wt.)	5869	1487	No	Ring 1&2_34 days	-134

Table 8.2 Comparison Between Study Mixtures and Class H and HT Specification Requirements

Mixture Number	Mixture Identification	56-day f'c >5175psi* (psi)	56-day Permeability <2,000 Coulombs Passed	Crack at 14-Days	Maximum Strain (microstrain)
1	0.38/6.8/FA20/SF5/II	Yes	Yes	No	-127
2	0.42/6.2/FA16/SF3.5/II	No	Yes	No	-112
3	0.38/6.8/FA20/SF5/G	Yes	Yes	No	-89
4	0.42/6.2/FA16/SF3.5/G	No	Yes	No	-94
5	0.44/6.5/FA30/II	Yes	Yes	No	-103
6	0.44/6.5/FA30/SF5/II	No	Yes	No	-96
7	0.44/6.5/BFS50/II	Yes	Yes	No	-113
8	0.44-6.0-FA30-SRA-II	Yes	Yes	No	-73
9	0.44-6.0-FA30-RET-II	Yes	Yes	No	-125
10	0.42-6.0-II (L.W.A)	Yes	Yes	No	-125
11	0.42-6.0-II (Normal Wt.)	Yes	Yes	No	-134

*Note: The 56 day required compressive strength of 4500psi is multiplied by 115% to account for laboratory settings.

8.3 Recommendations

A summary of recommended adjustments to the current CDOT Class H and HT structural concrete follows:

- Increase maximum allowable w/cm from 0.42 to 0.44;
- Increase maximum allowable cement replacement with Class F fly ash from 20-30%;
- Incorporate the use of cement replacement with ground-granulated blast furnace slag up to 50%;
- Incorporate the use of a shrinkage reducing admixture at high dosage rates;
- Incorporate the use of a set retarder admixture at average dosage rates;
- Decrease cementitious content to 564 lb/cy when supplementary cementitious materials are not used.

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APPENDIX A – MIXTURE DESIGNS

Mixture #1 (0.38-6.8-FA20-SF5-II) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	480	2.44	0.090
Fly Ash	128	0.87	0.032
BFS	0	0.00	0.000
Silica Fume	32	0.23	0.009
Rock	1766	10.84	0.402
Sand	1143	6.96	0.258
Water	243	3.90	0.144
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Content
0.275

Mix Characteristics

w/c	0.38
Unit Weight (pcf)	140.4
Cementitious material (lb)	640
Aggregate Volume (%)	66

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	20	128
BFS replacement (%)	0	0
SF replacement (%)	5	32

Moisture Content

sand pan	1072.2	sand + pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149

sand mc (%)	0.66	mc-ssd	-0.0004
rock mc (%)	0.08	mc-ssd	-0.00725

Batch Weights (yd^3)

Cement	480	lb
Fly Ash	128	lb
BFS	0	lb
Silica Fume	32	lb
Rock	1753	lb
Sand	1142	lb
Water	256	lb
HRWR / AEA	1.5	fl oz./cwt
HRWR / AEA	213	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00

Total	0.95
x 1.1	3.00

Batch Weights (ft^3)

Batchsize	3.0	cf
Cement	53.3	lb
Fly Ash	14.2	lb
BFS	0.0	lb
Silica Fume	3.6	lb
Rock	194.8	lb
Sand	126.9	lb
Water	28.5	lb
AEA	23.7	ml
HRWR	250	ml

Mixture #2 (0.42-6.2-FA16-SF3.5-II) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	467	2.38	0.088
Fly Ash	93	0.63	0.023
BFS	0	0.00	0.000
Silica Fume	20	0.15	0.005
Rock	1766	10.84	0.402
Sand	1206	7.35	0.272
Water	244	3.90	0.145
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.261

Mix Characteristics

w/c	0.42
Unit Weight (pcf)	140.6
Cementitious material (lb)	580
Aggregate Volume (%)	67

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	16	93
BFS replacement (%)	0	0
SF replacement (%)	3.5	20

Moisture Content

sand pan	1072.2	sand + pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	0.66	mc-ssd	-0.0004
rock mc (%)	0.08	mc-ssd	-0.00725

Batch Weights (yd³)

Cement	467	lb
Fly Ash	93	lb
BFS	0	lb
Silica Fume	20	lb
Rock	1753	lb
Sand	1205	lb
Water	257	lb
HRWR / AEA	1.5	fl oz./cwt
HRWR / AEA	257	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
Total		0.95
x 1.1		3.00

Batch Weights (ft³)

Batchsize	3.0	cf
Cement	51.9	lb
Fly Ash	10.3	lb
BFS	0.0	lb
Silica Fume	2.3	lb
Rock	194.8	lb
Sand	133.9	lb
Water	28.5	lb
AEA	28.6	ml
HRWRA	100	ml

Mixture #3 (0.38-6.8-FA20-SF5-G) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	480	2.44	0.090
Fly Ash	128	0.87	0.032
BFS	0	0.00	0.000
Silica Fume	32	0.23	0.009
Rock	1766	10.84	0.402
Sand	1143	6.96	0.258
Water	243	3.90	0.144
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.275

Mix Characteristics

w/c	0.38
Unit Weight (pcf)	140.4
Cementitious material (lb)	640
Aggregate Volume (%)	66

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	20	128
BFS replacement (%)	0	0
SF replacement (%)	5	32

Moisture Content

sand pan	1072.2	sand + pan wt.	1910
rock pan	1165.1	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149

sand mc (%)	0.88	mc-ssd	0.0018
rock mc (%)	3.85	mc-ssd	0.0305

Batch Weights (yd^3)

Cement	480	lb
Fly Ash	128	lb
BFS	0	lb
Silica Fume	32	lb
Rock	1820	lb
Sand	1145	lb
Water	187	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	142	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
Total		2.50
x 1.2		3.00

Batch Weights (ft^3)

Batch size	3.0	cf
Cement	53.3	lb
Fly Ash	14.2	lb
BFS	0.0	lb
Silica Fume	3.6	lb
Rock	202.2	lb
Sand	127.2	lb
Water	20.8	lb
HRWR / AEA	15.8	ml

Mixture #4 (0.42-6.2-FA16-SF3.5-G) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	467	2.38	0.088
Fly Ash	93	0.63	0.023
BFS	0	0.00	0.000
Silica Fume	20	0.15	0.005
Rock	1766	10.84	0.402
Sand	1206	7.35	0.272
Water	244	3.90	0.145
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.261

Mix Characteristics

w/c	0.42
Unit Weight (pcf)	140.6
Cementitious material (lb)	580
Aggregate Volume (%)	67

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	16	93
BFS replacement (%)	0	0
SF replacement (%)	3.5	20

Moisture Content

sand pan	1072.2	sand+pan wt.	1910
rock pan	1027.7	rock+pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	0.80	mc-ssd	0.001
rock mc (%)	0.10	mc-ssd	-0.007

Batch Weights (yd^3)

Cement	467	lb
Fly Ash	93	lb
BFS	0	lb
Silica Fume	20	lb
Rock	1754	lb
Sand	1207	lb
Water	255	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	138	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
Total		2.75
x 1.15		3.16

Batch Weights (ft^3)

Batch size	3.2	cf
Cement	54.7	lb
Fly Ash	10.9	lb
BFS	0.0	lb
Silica Fume	2.4	lb
Rock	205.4	lb
Sand	141.4	lb
Water	29.8	lb
HRWR / AEA	16.2	ml

Mixture #5 (0.44-6.5-FA30-II) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	428	2.18	0.081
Fly Ash	183	1.24	0.046
BFS	0	0.00	0.000
Silica Fume	0	0.00	0.000
Rock	1766	10.84	0.402
Sand	1096	6.68	0.247
Water	269	4.31	0.160
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.286

Mix Characteristics

w/c	0.44
Unit Weight (pcf)	138.6
Cementitious material (lb)	611
Aggregate Volume (%)	65

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	30	183
BFS replacement (%)	0	0
SF replacement (%)	0	0

Moisture Content

sand pan	1072.2	sand+pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149

sand mc (%)	0.80	mc-ssd	0.001
rock mc (%)	0.10	mc-ssd	-0.007

Batch Weights (yd^3)

Cement	428	lb
Fly Ash	183	lb
BFS	0	lb
Silica Fume	0	lb
Rock	1754	lb
Sand	1097	lb
Water	280	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	126	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00

Total	2.75
x 1.15	3.16

Batch Weights (ft^3)

Batch size	3.2	cf
Cement	50.1	lb
Fly Ash	21.5	lb
BFS	0.0	lb
Silica Fume	0.0	lb
Rock	205.4	lb
Sand	128.5	lb
Water	32.8	lb
HRWR / AEA	14.8	ml

Mixture #6 (0.44-6.5-FA30-SF5-II) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	397	2.02	0.075
Fly Ash	183	1.24	0.046
BFS	0	0.00	0.000
Silica Fume	31	0.22	0.008
Rock	1766	10.84	0.402
Sand	1085	6.61	0.245
Water	269	4.31	0.160
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.289

Mix Characteristics

w/c	0.44
Unit Weight (pcf)	138.2
Cementitious material (lb)	611
Aggregate Volume (%)	65

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	30	183
BFS replacement (%)	0	0
SF replacement (%)	5	31

Moisture Content

sand pan	1072.2	sand + pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	0.10	mc-ssd	-0.006
rock mc (%)	0.20	mc-ssd	-0.006

Batch Weights (yd^3)

Cement	397	lb
Fly Ash	183	lb
BFS	0	lb
Silica Fume	31	lb
Rock	1755	lb
Sand	1078	lb
Water	286	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	117	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
Total		2.75
x 1.2		3.30

Batch Weights (ft^3)

Batch size	3.3	cf
Cement	48.5	lb
Fly Ash	22.4	lb
BFS	0.0	lb
Silica Fume	3.7	lb
Rock	214.5	lb
Sand	131.8	lb
Water	34.9	lb
HRWR / AEA	14.4	ml

Mixture #7 (0.44-6.5-BFS50-II) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	306	1.55	0.058
Fly Ash	0	0.00	0.000
BFS	306	1.69	0.063
Silica Fume	0	0.00	0.000
Rock	1766	10.84	0.402
Sand	1124	6.85	0.254
Water	269	4.31	0.160
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.280

Mix Characteristics

w/c	0.44
Unit Weight (pcf)	139.6
Cementitious material (lb)	611
Aggregate Volume (%)	66

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	50	306
SF replacement (%)	0	0

Moisture Content

sand pan	1072.2	sand + pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	0.10	mc-ssd	-0.006
rock mc (%)	0.20	mc-ssd	-0.006

Batch Weights (yd^3)

Cement	306	lb
Fly Ash	0	lb
BFS	306	lb
Silica Fume	0	lb
Rock	1755	lb
Sand	1118	lb
Water	286	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	90	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
	Total	2.75
	x 1.2	3.30

Batch Weights (ft^3)

Batch size	3.3	cf
Cement	37.3	lb
Fly Ash	0.0	lb
BFS	37.3	lb
Silica Fume	0.0	lb
Rock	214.5	lb
Sand	136.6	lb
Water	35.0	lb
HRWR / AEA	11.0	ml

Mixture #8 (0.44-6.0-FA30-SRA-II) Concrete Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	378	1.92	0.071
Fly Ash	162	1.10	0.041
BFS	0	0.00	0.000
Silica Fume	0	0.00	0.000
Rock	1766	10.84	0.402
Sand	1243	7.58	0.281
Water	238	3.81	0.141
Air	0.065	1.76	0.065
SRA (fl. oz/cu	2.0	27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fume	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.253

Mix Characteristics

w/c	0.44
Unit Weight (pcf)	140.3
Cementitious material (lb)	540
Aggregate Volume (%)	68

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	30	162
BFS replacement (%)	0	0
SF replacement (%)	0	0

Shrinkage Reducing Admixture Dosage

Min. Dosage -	0.5	Gal / yd3	Max. Dosa	1.5	Gal / yd3			
S.R.A.	0.1	Gal / batch	S.R.A.	0.2	Gal / bat	1 gal -	3785.4	mL
	245.4	mL / batch		736.1	mL / batch			

Moisture Content

sand pan	1072.2	sand + pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	0.82	mc-ssd	0.0012
rock mc (%)	0.11	mc-ssd	-0.0069

Batch Weights (yd^3)

Cement	378	lb
Fly Ash	162	lb
BFS	0	lb
Silica Fume	0	lb
Rock	1754	lb
Sand	1245	lb
Water	248	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	112	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
Total		1.10
x 1.1		3.50

Batch Weights (ft^3)

Batch size	3.5	cf
Cement	49.0	lb
Fly Ash	21.0	lb
BFS	0.0	lb
Silica Fume	0.0	lb
Rock	227.3	lb
Sand	161.4	lb
Water	32.2	lb
HRWR / AEA	14.5	ml

Mixture #9 (0.44-6.0-FA30-RET-II) Concrete Mixture Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	378	1.92	0.071
Fly Ash	162	1.10	0.041
BFS	0	0.00	0.000
Silica Fume	0	0.00	0.000
Rock	1766	10.84	0.402
Sand	1243	7.58	0.281
Water	238	3.81	0.141
Air	0.065	1.76	0.065
RET (fl. oz/cwt	3.0	27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand	2.63	0.70

Paste Volume
0.253

Mix Characteristics

w/c	0.44
Unit Weight (pcf)	140.3
Cementitious material (lb)	540
Aggregate Volume (%)	68

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	30	162
BFS replacement (%)	0	0
SF replacement (%)	0	0

Moisture Content

sand pan	1072.2	sand + pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	0.84	mc-ssd	0.0014
rock mc (%)	0.23	mc-ssd	-0.0057

Batch Weights (yd^3)

Cement	378	lb
Fly Ash	162	lb
BFS	0	lb
Silica Fume	0	lb
Rock	1756	lb
Sand	1245	lb
Water	246	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	112	ml

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
Total		1.10
x 1.1		3.50

Batch Weights (ft^3)

Batchsize	3.5	cf
Cement	49.0	lb
Fly Ash	21.0	lb
BFS	0.0	lb
Silica Fume	0.0	lb
Rock	227.6	lb
Sand	161.4	lb
Water	31.9	lb
HRWR / AEA	14.5	ml

Set Retarder Dosage				
Dosage - 3oz.cwt			1oz-29.57mL	
2.1	oz		62.1	mL

Mixture #10 (0.42-6.0-II (L.W.A.)) Concrete Mixture Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	564	2.87	0.106
Fly Ash	0	0.00	0.000
BFS	0	0.00	0.000
Silica Fume	0	0.00	0.000
Rock	1766	10.84	0.402
Sand	906	5.52	0.205
Sand LWA	250	2.21	0.082
Water	237	3.80	0.141
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand lwa	1.81	18.71
Sand	2.63	0.7

Paste Volume
0.247

Mix Characteristics

w/c	0.42
Unit Weight (pcf)	137.9
Cementitious material (lb)	564
Aggregate Volume (%)	69

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	0	0
SF replacement (%)	0	0

Moisture Content (Traditional Rock and Sand)

sand pan	1072.2	sand + pan wt.	1910
rock pan	1027.7	rock + pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	0.72	mc-ssd	0.0002
rock mc (%)	0.10	mc-ssd	-0.007

Moisture Content (LWA Sand)

sand pan	1072.2	sand + pan wt.	1910
		dry wt. sand	1875
sand mc (%	18.36	mc-ssd	-0.0035

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
	Total	0.95
	x 1.1	3.35

Batch Weights (yd³)

Cement	564	lb
Fly Ash	0	lb
BFS	0	lb
Silica Fume	0	lb
Rock	1754	lb
Sand	907	lb
Sand LWA	249	lb
Water	250	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	167	ml

Batch Weights (ft³)

Batchsize	3.4	cf
Cement	70.0	lb
Fly Ash	0.0	lb
BFS	0.0	lb
Silica Fume	0.0	lb
Rock	217.6	lb
Sand	112.5	lb
Sand LWA	30.9	lb
Water	31.0	lb
AEA	20.7	mL
HRWRA	42	mL

Mixture #11 (0.42-6.0-II(Norm.Wt.)) Concrete Mixture Design Spreadsheet

Mix Proportion (SSD)

Material	Weight (lb/cy)	Volume (cf)	Volume Check
Cement	564	2.87	0.106
Fly Ash	0	0.00	0.000
BFS	0	0.00	0.000
Silica Fume	0	0.00	0.000
Rock	1766	10.84	0.402
Sand	1270	7.74	0.287
Sand LWA	0	0.00	0.000
Water	237	3.80	0.141
Air	0.065	1.76	0.065
		27.00	1.00

Material Properties

Material	S.G.	A.C
Cement	3.15	-
Fly Ash (F)	2.37	-
BFS	2.90	-
Silica Fum	2.20	-
Rock	2.61	0.80
Sand lwa	1.81	18.71
Sand	2.63	0.7

Paste Volume
0.247

Mix Characteristics

w/c	0.42
Unit Weight (pcf)	142.1
Cementitious material (lb)	564
Aggregate Volume (%)	69

Suppl. Cementitious Mat.	Percent (%)	Weight (lb)
Fly Ash replacement (%)	0	0
BFS replacement (%)	0	0
SF replacement (%)	0	0

Moisture Content (Traditional Rock and Sand)

sand pan	1072.2	sand+pan wt.	1910
rock pan	1027.7	rock+pan wt	2178.2
		dry wt. sand	1875
		dry wt. rock	2149
sand mc (%)	1.15	mc-ssd	0.0045
rock mc (%)	0.45	mc-ssd	-0.0035

Moisture Content (LWA Sand)

sand pan	1072.2	sand+pan wt.	1910
		dry wt. sand	1875
sand mc (%	18.36	mc-ssd	-0.0035

Testing Specimens Required

Compressive cylinders	12	0.70
RCIP cylinders	0	0.00
MOR	0	0.00
Unit weight	1	0.25
Permeameter slabs	0	0.00
Salt Ponding	0	0.00
MOE	0	0.00
Beam Molds	0	0.00
Split Cylinder	0	0.00
	Total	0.95
	x 1.1	3.35

Batch Weights (yd^3)

Cement	564	lb
Fly Ash	0	lb
BFS	0	lb
Silica Fume	0	lb
Rock	1760	lb
Sand	1275	lb
Sand LWA	0	lb
Water	237	lb
HRWR / AEA	1.0	fl oz./cwt
HRWR / AEA	167	ml

Batch Weights (ft^3)

Batchsize	3.4	cf
Cement	70.0	lb
Fly Ash	0.0	lb
BFS	0.0	lb
Silica Fume	0.0	lb
Rock	218.3	lb
Sand	158.2	lb
Sand LWA	0.0	lb
Water	29.4	lb
AEA	20.7	mL
HRWR	251	mL

APPENDIX B - MATERIALS PRODUCT DATA

LABORATORY TEST REPORT

WesTest
Specialists in the Paving Industry
845 Navajo Street
Denver, CO 80204
303.975.9959, Fax 303.975.9969

CLIENT: Bestway Concrete
SOURCE: Brighton
SAMPLED BY: Client
PROJECT: Miscellaneous

WesTest PROJECT NO.: 202408
REPORT DATE: March 5, 2008

MATERIAL DESCRIPTION	ASTM C 33 Fine Aggregate
DATE SAMPLED	January 28, 2008
SAMPLE LOCATION	Stockpile

Aggregate Physical Property and Quality Tests (ASTM C 33, AASHTO M 6 Specifications)

ASTM C 117 & C 136, AASHTO T 11 & T 27

SIEVE SIZE	% Passing	ASTM C 33 Spec.	AASHTO M 6 Spec.
1"			
3/4"			
1/2"			
3/8"		100	100
# 4	100	95 - 100	95 - 100
# 8	99	80 - 100	80 - 100
# 16	71	50 - 85	50 - 85
# 30	38	25 - 60	25 - 60
# 50	14	5 - 30	5 - 30
# 100	4	0 - 10	0 - 10
# 200	1.4	0 - 3	0 - 2
Fineness Modulus	2.74	2.3 - 3.1	

ASTM C 128, AASHTO T 84, Bulk Specific Gravity = 2.61, Bulk Specific Gravity (SSD) = 2.63, Apparent Specific Gravity = 2.66, Absorption = 0.7%

ASTM D 2419, AASHTO T 176,
Sand Equivalent Value = 90
Specification: 80 Min. (CDOT)

ASTM C 142, AASHTO T 112, Clay Lumps & Friable Particles
FINE AGG. = 0.7%, Specification: 3.0% Max.

ASTM C 123, AASHTO T 113, Lightweight Particles in Aggregate

SAMPLE WT.	LIQUID TYPE / SPECIFIC GRAVITY	LIGHTWEIGHT PARTICLES	SPEC.
210.1	$ZnCl_2$/2.0	0.0%	0.5% Max.
210.1	$ZnBr_2$/2.4	0.0%	3.0% Max.

ASTM C 88, AASHTO T 104, Sodium Sulfate Soundness, 5 Cycles

SIEVE SIZE	GRADING OF ORIGINAL SAMPLE	WEIGHT BEFORE TEST, g	PERCENT PASSING AFTER TEST	WEIGHTED PERCENT LOSS
Minus #100	4			
# 50 to # 100	10			
# 30 to # 50	24	100.0	4.0	1.0
# 16 to # 30	33	100.0	1.2	0.4
# 8 to # 16	28	100.0	2.0	0.6
# 4 to # 8	1		2.0	0.0
3/8" to # 4				
TOTAL	100	FINE AGG. TOTAL 100%		2
SPECIFICATION:				10 Max.

ASTM C 40, AASHTO T 21, Organic Impurities:
Less than Organic Plate No. 1
Specification: Organic Plate No. 3 or Less

COMMENTS:

TABLE 1

WesTest — Specialists to the Paving Industry
845 Navajo Street
Denver, CO 80204
303.975.9959, Fax 303.975.9969

LABORATORY TEST REPORT

CLIENT: Bestway Concrete
SOURCE: Brighton
SAMPLED BY: Client
PROJECT: Miscellaneous

WesTest PROJECT NO.: 202408
REPORT DATE: March 5, 2008

MATERIAL DESCRIPTION	ASTM C 33 Size No. 57/67 Coarse Aggregate
DATE SAMPLED	January 28, 2008
SAMPLE LOCATION	Stockpile

Aggregate Physical Property and Quality Tests (ASTM C 33 Specifications)

ASTM C 117 & C 136, AASHTO T 11 & T 27

SIEVE SIZE	% Passing	No. 57 Specification	No. 67 Specification
2"			
1-1/2"	100	100	
1"	100	95 - 100	100
3/4"	92		90 - 100
1/2"	60	25 - 60	
3/8"	40		20 - 55
# 4	7	0 - 10	0 - 10
# 8	3	0 - 5	0 - 5
# 16	2		
# 30	2		
# 50	1		
# 100	1		
# 200	0.9	0 - 1.5	0 - 1.5

ASTM C 127, AASHTO T 85, Bulk Specific Gravity = 2.59, Bulk Specific Gravity (SSD) = 2.61, Apparent Specific Gravity = 2.64, Absorption = 0.8%

ASTM C 131, AASHTO T 96, L.A. Abrasion
Grading B, Loss = 40%
Specification: 45% Max.

ASTM C 142, AASHTO T 112, Clay Lumps & Friable Particles
COARSE AGG. = 0.2%, Specification: 3.0% MAX.

ASTM C 123, AASHTO T 113, Lightweight Particles in Aggregate

SAMPLE WT. (g)	LIQUID TYPE / SPECIFIC GRAVITY	LIGHTWEIGHT PARTICLES	SPEC.
3188.7	$ZnCl_2$/2.0	0.0%	0.5% Max.
3188.7	$ZnBr_2$/2.4	0.0%	3.0% Max.

ASTM C 88, AASHTO T 104, Sodium Sulfate Soundness, 5 Cycles

SIEVE SIZE	GRADING OF ORIGINAL SAMPLE	WEIGHT BEFORE TEST, g	PERCENT PASSING AFTER TEST	WEIGHTED PERCENT LOSS
1-1/2" to 1"	9		1.6	0.1
1" to 3/4"		514.7		
3/4" to 1/2"	56	672.8	0.6	0.4
1/2" to 3/8"		331.4		
3/8" to No.4	35	300.7	0.3	0.1
TOTAL	100	COARSE AGG. TOTAL 5%		1
SPECIFICATION:				12 Max.

ASTM C 29, AASHTO T 19,
Bulk Density and Voids in Aggregate
Rodding Method: Bulk Density = 103 pcf
Voids in Aggregate = 36%

COMMENTS

TABLE 3

845 Navajo Street • Denver, CO 80204

SPECIALISTS TO THE PAVING INDUSTRY

Phone: 303.975.9959 • Fax: 303.975.9969 • Email:office@westest.net

March 11, 2008

Bestway Concrete
315 Frontier Court
Milliken, CO 80543

Attention: Mr. Dan Bentz

Subject: Laboratory Test Results
Brighton Pit
ASTM C 1260 and CP-L 4201 Potential Alkali Reactivity of Aggregates
ASTM C 33 Coarse Aggregate
ASTM C 33 Fine Aggregate
WesTest Project No. 202408

Gentlemen:

Enclosed as Figures 1 and 2 are the results of potential alkali reactivity testing (mortar bar method), preformed on aggregate sampled from the above-referenced source on January 28, 2008. The aggregate was prepared and tested in general accordance with Colorado and/or ASTM Procedures. ASTM C 1260 defines the potential of an aggregate for deleterious expansion as follows:

Test Expansion	Classification	Potential for Deleterious ASR
< 0.10%	Innocuous	Low
0.10% to 0.20%	Inconclusive	Not Predictable
> 0.20%	Deleterious	High

Based on the test results of 0.06% expansion at 14 days in solution, 16 days after casting, the potential for deleterious alkali-silica behavior of this aggregate in concrete is considered low.

If you have any questions on the data presented, please contact us at your convenience.

Sincerely,
WesTest

John J. Cessar, EI

Reviewed by:
WesTest

Eric R. West, P.E.

WESTEST
SPECIALISTS TO THE PAVING INDUSTRY
545 Navajo Street
Denver, CO 80204
303.975.9959

LABORATORY TEST REPORT

POTENTIAL ALKALI REACTIVITY OF AGGREGATES
(MORTAR-BAR METHOD)
ASTM C 1260 / CP-L 4201

CLIENT: Bestway Concrete
PROJECT NO.: 202408

REPORT DATE: March 11, 2008

SAMPLE ID: Brighton Coarse Aggregate

AGGREGATE:
SOURCE: Brighton Pit
SIZE: ASTM C 33 Coarse Aggregate

COMMENTS: Aggregate graded as per Section 7.2, Table 1

CEMENT:
SOURCE: Holcim
TYPE: I/II GU
AUTOCLAVE EXPANSION: 0.02%
ALKALIS CONTENT (as Na equivalent): 0.75%
COMMENTS: Cement data provided by Holcim

MIX WATER:
0.47 w/c ratio

EFFECTIVE GAUGE LENGTH = 250 mm

Specimen	2/20/08 Initial	2/21/08 Zero	2/25/08 4 Days		2/29/08 8 Days		3/3/08 11 Days		3/6/08 14 Days	
	Comparator Reading	Comparator Reading	Comparator Reading	Length Change	Comparator Reading	Length Change	Comparator Reading	Length Change	Comparator Reading	Length Change
A	-0.170	-0.010	0.026	0.01%	0.072	0.03%	0.082	0.04%	0.124	0.05%
B	-0.246	-0.090	-0.050	0.02%	-0.004	0.03%	-0.004	0.03%	0.058	0.06%
C	-0.180	-0.022	-0.010	0.00%	0.050	0.03%	0.050	0.03%	0.110	0.05%
AVERAGE		-0.041	-0.011	0.01%	0.039	0.03%	0.043	0.03%	0.097	0.06%

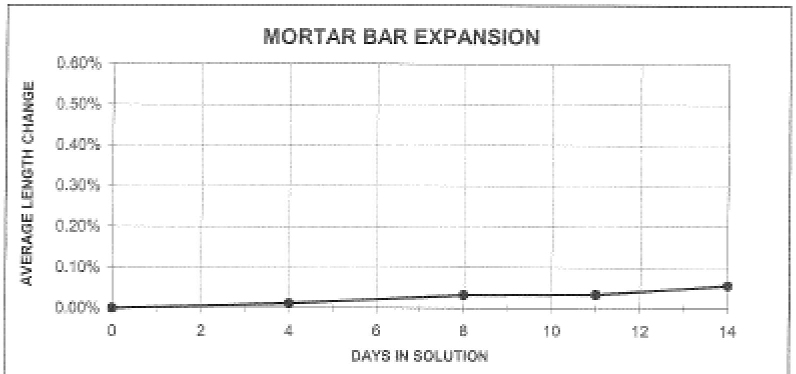

FIGURE 1

WesTest
Specialists to the Paving Industry
845 Navajo Street
Denver, CO 80204
303.975.9959

LABORATORY TEST REPORT

POTENTIAL ALKALI REACTIVITY OF AGGREGATES
(MORTAR-BAR METHOD)
ASTM C 1260 / CP-L 4201

CLIENT: Bestway Concrete
PROJECT NO.: 202408

REPORT DATE: March 11, 2008

SAMPLE ID: Brighton Fine Aggregate

AGGREGATE:
SOURCE: Brighton Pit
SIZE: ASTM C 33 Fine Aggregate

COMMENTS: Aggregate graded as per Section 7.2, Table 1

CEMENT:
SOURCE: Holcim
TYPE: I/II GU
AUTOCLAVE EXPANSION: 0.02%
ALKALIS CONTENT (as Na equivalent): 0.75%
COMMENTS: Cement data provided by Holcim

MIX WATER:
0.47 w/c ratio

EFFECTIVE GAUGE LENGTH = 250 mm

	2/22/08	2/23/08	2/26/08		2/29/08		3/4/08		3/8/08	
	Initial	Zero	3 Days		6 Days		10 Days		14 Days	
Specimen	Comparator Reading	Comparator Reading	Comparator Reading	Length Change	Comparator Reading	Length Change	Comparator Reading	Length Change	Comparator Reading	Length Change
A	-0.212	-0.042	-0.014	0.01%	0.030	0.03%	0.072	0.05%	0.096	0.06%
B	-0.200	-0.038	-0.014	0.01%	0.042	0.03%	0.088	0.05%	0.114	0.06%
C	-0.174	-0.012	0.022	0.01%	0.072	0.03%	0.118	0.05%	0.142	0.06%
AVERAGE		-0.031	-0.002	0.01%	0.048	0.03%	0.093	0.05%	0.117	0.06%

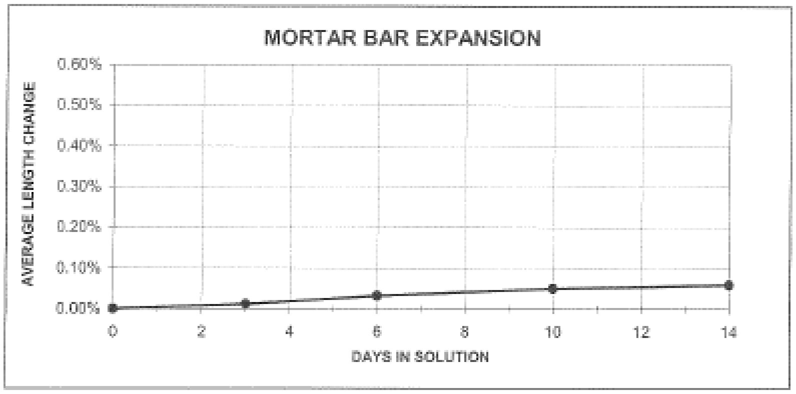

FIGURE 2

Task/Type Standard/Program: 13NUCMT/T09CMT/09_NST2
Date and Time of Analysis: 7/16/2009 8:53:48 AM
Type of Analysis: Concentration Analysis
Number of Repeats: 1
Cassette Number: 43

K=1 M2=2 M3=3 OT=4 I-II MA Sample Number: PRETEST

12 - HMA
W. of Denver

Run	Si	Al	Fe	Ca	Mg	S	Na	K	Cl	L550	L950	Total	NaEQ	C3S	C2S
Avg	19.34	4.78	3.33	63.55	1.37	3.88	0.243	0.897	0.098	1.00	2.50	99.99	0.83	56.02	13.20

	C3A	C4AF	SSCAF	CAF2CA	AF	CO2	LSCO2	LIMSTN	D1
Avg	7.01	10.15	0.00	24.17	1.43	1.50	35.00	4.3	0.00

420 Blaine
1.74 Free lime

K. Kane

Boral Material Technologies

BORAL

ASTM C 618 TEST REPORT

Sample Number:	S-070613011	Report Date:	7/27/2007
Sample Dates:	May 2007	Sample Source:	Pawnee
		Tested By:	KP

TESTS	RESULTS	ASTM C 618 CLASS F/C	AASHTO M 29 CLASS F/C
C618 QC			
CHEMICAL TESTS			
Silicon Dioxide (SiO_2), %	30.14		
Aluminum Oxide (Al_2O_3), %	18.73		
Iron Oxide (Fe_2O_3), %	5.76		
Sum of SiO_2, Al_2O_3, Fe_2O_3, %	54.63	70.0/50.0 min.	70.0/50.0 min.
Calcium Oxide (CaO), %	28.54		
Magnesium Oxide (MgO), %	7.66		
Sulfur Trioxide (SO_3), %	2.74	5.0 max.	5.0 max.
Sodium Oxide (Na_2O), %	2.07		
Potassium (K_2O), %	0.33		
Total Alkalies (as Na_2O), %	2.29		
Available Alkalies (as Na_2O), %			
PHYSICAL TESTS			
Moisture Content, %	0.03	3.0 max.	3.0 max.
Loss on Ignition %	0.26	6.0 max.	6.0 max.
Amount Retained on No. 325 Sieve, %	14.55	34 max.	5.0 max.
Specific Gravity	2.79		
Autoclave Soundness, %	0.01	0.8 max.	0.8 max.
SAI, with Portland Cement at 7 days, % of Control	96.8	75 min.*.	75 min.*.
SAI, with Portland Cement at 28 days, % of Control	98.8	75 min.*.	75 min.*.
Water Required, % of Control	93.4	105 max.	105 max.
Bulk Density			

Meets ASTM C 618 and AASHTO M 295, Class C

The Class (C) Fly Ash from this plant meets the requirements of the MDOT and SCDHPT specifications.

* Meeting the 7 day or 28 day Strength Activity Index will indicate specification compliance.

Approved By: Diana Benfield (signature)
Diana Benfield
QC Specialist

Approved By: Brian Shaw (signature)
Brian Shaw
Materials Testing Manager

45 NE LOOP 410, SUITE 700 SAN ANTONIO, TEXAS 210.349.4069

Boral Material Technologies

ASTM C 618 TEST REPORT

Sample Number: S-070625011
Sample Dates: June 2007

Report Date: 8/10/2007
Sample Source: Micron3 Rockdale
Tested By: MER

TESTS	RESULTS	ASTM C 618 CLASS F/C	AASHTO M 295 CLASS F/C
C618 QC			
CHEMICAL TESTS			
Silicon Dioxide (SiO2), %	55.03		
Aluminum Oxide (Al2O3), %	24.48		
Iron Oxide (Fe2O3), %	4.24		
Sum of SiO2, Al2O3, Fe2O3, %	83.75	70.0/50.0 min.	70.0/50.0 min.
Calcium Oxide (CaO), %	9.73		
Magnesium Oxide (MgO), %	1.78		
Sulfur Trioxide (SO3), %	0.81	5.0 max.	5.0 max.
Sodium Oxide (Na2O), %	0.57		
Potassium (K2O), %	0.99		
Total Alkalies (as Na2O), %	1.22		
Available Alkalies (as Na2O), %			
PHYSICAL TESTS			
Moisture Content, %	0.09	3.0 max.	3.0 max.
Loss on Ignition %	0.29	6.0 max.	5.0 max.
Amount Retained on No. 325 Sieve, %	0.68	34 max.	34.0 max.
Specific Gravity	2.57		
Autoclave Soundness, %	-0.10	0.8 max.	0.8 max.
SAI, with Portland Cement at 7 days, % of Control	101.0	75 min.*.	75 min.*.
SAI, with Portland Cement at 28 days, % of Control	130.5	75 min.*.	75 min.*.
Water Required, % of Control	89.3	105 max.	105 max.
Bulk Density			

Meets ASTM C 618 and AASHTO M 295, FDOT Section 929, SCDHPT and MDOT specifications for Class F Fly Ash

* Meeting the 7 day or 28 day Strength Activity Index will indicate specification compliance.

Approved By: Diana Benfield
Diana Benfield
QC Specialist

Approved By: Brian Shaw
Brian Shaw
Materials Testing Manager

45 NE LOOP 410, SUITE 700 SAN ANTONIO, TEXAS 210.349.4069

Grace Concrete Products

GRACE

DARACEM® 19

High-range water-reducing admixture

ASTM C494 Type A and F, and ASTM C1017 Type I

Product Description

Daracem® 19 is an aqueous solution of a modified naphthalene sulfonate. Daracem 19 is a superior dispersing admixture having a marked capacity to disperse the cement agglomerates normally found in a cement-water suspension. The capability of Daracem 19, in this respect, exceeds that of normal water-reducing admixtures. It is a low viscosity liquid manufactured for use as received. Daracem 19 contains no added chloride. Daracem 19 is formulated to comply with *Specifications for Chemical Admixtures for Concrete*, ASTM C494 as a Type A and Type F admixture, and ASTM C1017 as a Type I admixture. One gallon of Daracem 19 weighs approximately 10 lbs (1.2 kg/L).

Uses

Daracem 19 produces concrete with extremely workable characteristics referred to as high slump. Daracem 19 also allows concrete to be produced with very low water/cement ratios at low or normal slumps.

Daracem 19 is ideal for use in prestress, precast, bridge deck or any concrete where it is desired to keep the water/cement ratio to a minimum and still achieve the degree of workability necessary to provide easy placement and consolidation. Daracem 19 will also fluidize concrete, making it ideal for tremie concreting or other applications where high slumps are desired.

Addition Rates

Addition rates of Daracem 19 can vary with type of application, but will normally range from 6 to 20 fl oz/100 lbs (390 to 1300 mL/100 kg) of cement. In most instances the addition of 10 to 16 fl oz/100 lbs (650 to 1040 mL/100 kg) of cement will be sufficient. At a given water/cement ratio, the slump required for placement can be controlled by varying the addition rate. Should job site conditions require using more than recommended addition rates, please consult your Grace representative.

Product Advantages

- Can produce high slump flowable concrete with no loss in strength
- Can produce low water/cement ratio concrete and therefore, high strengths
- Concrete produced with Type I cement may be substituted for normal concrete produced with Type III cement to achieve early strengths
- At high slump, exhibits no significant segregation in comparison to concrete without a superplasticizer at the same slump

Compatibility with Other Admixtures and Batch Sequencing

Daracem 19 is compatible with most Grace admixtures as long as they are added separately to the concrete mix, usually through the water holding tank discharge line. However, Daracem 19 is not recommended for use in concrete containing ADVA® superplasticizers or MIRA® 92. In general, it is recommended that Daracem 19 be added to the concrete mix near the end of the batch sequence for optimum performance. Different sequencing may be used if local testing shows better performance. Please see Grace Technical Bulletin TB-0110, *Admixture Dispenser Discharge Line Location and Sequencing for Concrete Batching Operations* for further recommendations. Daracem 19 should not come in contact with any other admixture before or during the batching process, even if diluted in mix water.

Pretesting of the concrete mix should be performed before use, and as conditions and materials change in order to assure compatibility, and to optimize dosage rates, addition times in the batch sequencing and concrete performance. For concrete that requires air entrainment, the use of an ASTM C260 air-entraining agent (such as Daravair® or Darex® II AEA) is recommended to provide suitable air void parameters for freeze-thaw resistance. Darex AEA is not recommended. Please consult your Grace representative for guidance.

Packaging & Handling

Daracem 19 is available in bulk, delivered by metered tank trucks, and in 55 gal (210 L) drums.

It will begin to freeze at approximately 32°F (0°C), but will return to full strength after thawing and thorough agitation.

In storage, and for proper dispensing, Daracem 19 should be maintained at temperatures above 32°F (0°C).

Dispensing Equipment

A complete line of accurate, automatic dispensing equipment is available.

www.graceconstruction.com

North American Customer Service: 1-877-4AD-MIX1 (1-877-423-6491)

DC-8 Printed in U.S.A. 11/07 FA/LI/1M

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Grace Concrete Products

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DARAVAIR® AT60

Air-entraining admixture

ASTM C260

Product Description

Daravair® AT60 is a liquid air-entraining admixture that provides freeze-thaw resistance, enhances the finishability characteristics of concrete, and allows concrete producers to accurately control yield. Daravair AT60 is comprised of a blend of high-grade saponified rosin and organic acid salts, and is manufactured under stringent controls, assuring quality and consistent performance.

Uses

Daravair AT60 is recommended for use in all ready-mix, precast, prestress and other concrete product plants where the intentional entrainment of a specified level of air is required. ACI 201 *Guide to Durable Concrete* recommends all concrete which is exposed to any level of freeze-thaw exposure or is subjected to the application of de-icing salts during the winter months should be air entrained.

Daravair AT60 has been found to be particularly effective in both high cement factor and low slump concrete mixes, which require a very efficient air-entraining admixture. Daravair AT60 is also often utilized when a very stable air void system over time is required.

Product Advantages

- Air stability makes it particularly useful for longer transit times
- Functions well across a wide range of concrete materials
- Economical to use in concretes which are typically difficult to air entrain

Performance

Air is incorporated into concrete via mixing mechanics and stabilized into millions of discrete semi-microscopic bubbles in the presence of air-entraining admixtures such as Daravair AT60. These air bubbles act much like flexible ball bearings, thereby increasing the plasticity and workability of the concrete. This allows for reductions in mixing water with no loss of slump. Surface bleeding, plastic shrinkage and aggregate segregation are also minimized.

Through the purposeful entrainment of air, Daravair AT60 markedly increases the durability of concrete to severe exposures, particularly freeze-thaw cycling. It has also demonstrated a remarkable ability to impart resistance to the action of frost and de-icing salts as well as sulfate, sea and alkaline waters.

Addition Rates

Daravair AT60 addition rates will vary according to the specified level of air required. Addition rates are also influenced by several variables including specific mix design parameters, material properties of the cement, fly ash, coarse and fine aggregates, and the effects of other chemical admixtures. Other factors such as ambient and concrete temperature, mixing time, and time of addition can also affect the required dosage rates. It is recommended that pre-job testing be conducted in order to assure the correct dosage rate of Daravair AT60 is used. Typical Daravair AT60 addition rates range from ¼ to 3 fl oz/100 lbs (15 to 200 mL/100 kg) of cement.

Compatibility with Other Admixtures and Batch Sequencing

Daravair AT60 is compatible with most Grace admixtures as long as they are added separately to the concrete mix. In general, it is recommended that Daravair AT60 be added to the concrete mix near the beginning of the batch sequence for optimum performance, preferably by "dribbling" on the sand. Different sequencing may be used if local testing shows better performance. Please see Grace Technical Bulletin TB-0110, *Admixture Dispenser Discharge Line Location and Sequencing for Concrete Batching Operations* for further recommendations. Daravair AT60 should not be added directly to heated water.

Pretesting of the concrete mix should be performed before use, and as conditions and materials change in order to assure compatibility, and to optimize dosage rates, addition times in the batch sequencing and concrete performance. Please consult your Grace representative for guidance.

Packaging & Handling

Daravair AT60 is available in bulk, delivered by metered tank truck, and in 55 gal (208 L) drums. Daravair AT60 should be protected from temperatures below 32°F (0°C), but if freezing does occur, thorough mechanical agitation after thawing will restore it to full strength. Adhere to MSDS guidelines when handling product.

Dispensing Equipment

A complete line of accurate dispensing equipment is readily available to dispense Daravair AT60. The dispensers can be installed to discharge the product into the water line, on the sand, or directly in the mixer.

Specifications

The concrete shall be intentionally air entrained, containing a specified level of entrained air. The plastic air content shall be determined by ASTM C231 pressure method, or ASTM C138 gravimetric method. The air entrainment admixture shall be Daravair AT60, as manufactured by Grace Construction Products, and will comply with ASTM C260 specification for air-entraining admixtures. The dosage rate of Daravair AT60 will be determined on a individual basis to satisfy the specified requirement for the particular job.

www.graceconstruction.com

North American Customer Service: 1-877-4AD-MIX1 (1-877-423-6491)

AT-1D Printed in U.S.A. 11/07 FA/LI/1M

GRACE

BASF

The Chemical Company

3 03 30 00 03 40 00 03 70 00	Product Data Cast-in-Place Concrete Precast Concrete Mass Concrete

TETRAGUARD® AS20

Shrinkage-Reducing Admixture

Description

Tetraguard AS20 shrinkage-reducing admixture is the first commercially available chemical admixture developed specifically to reduce drying shrinkage of concrete and mortar, and the potential for subsequent cracking. Tetraguard AS20 admixture has been used successfully in the Far East and North American construction markets since its introduction in 1985.

Tetraguard AS20 admixture was developed to replace/enhance inorganic expansive admixtures that were being used to prevent drying shrinkage cracking. These expansive admixtures acted by inducing compressive stresses in concrete to offset tensile stresses caused by drying shrinkage.

Tetraguard AS20 admixture functions by reducing capillary tension of pore water, a primary cause of drying shrinkage.

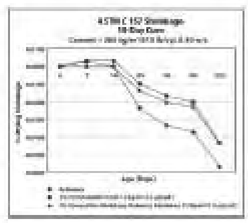
Applications

Recommended for use in:

- Ready-mixed or precast concrete structures requiring shrinkage reduction and long term durability
- Wet mix shotcrete
- Mortars and grouts

Features

- Significantly reduces drying shrinkage by as much as 80% at 28 days, and up to 50% at one year or beyond
- Reduces stresses induced from one-dimensional surface drying in concrete slabs and floors
- Reduces compressive creep
- Reduces carbonation



Benefits

- Reduces drying shrinkage cracking and microcracking thereby improving aesthetics, watertightness and durability
- Reduces compressive creep under drying conditions that minimizes prestress loss
- Minimizes curling

Performance Characteristics

Tetraguard AS20 admixture does not substantially affect slump. Tetraguard AS20 admixture may increase bleed time and bleed ratio (10% higher). Tetraguard AS20 admixture may also delay time of set by 1-2 hours depending upon dosage and temperature. Compressive strength loss is minimal with Tetraguard AS20 admixture.

All projects requiring Tetraguard AS20 admixture in concrete applications exposed to freezing and thawing environments must be pre-approved and require field trials prior to use. Therefore, contact your local sales representative when concrete treated with Tetraguard AS20 admixture is being proposed for applications exposed to freezing and thawing environments.

Guidelines for Use

Dosage: Knowledge of the shrinkage characteristics of the concrete mixture proposed for use is required prior to the addition of Tetraguard AS20 admixture. The dosage of Tetraguard AS20 admixture will be dependent on the desired drying shrinkage and the reduction in drying shrinkage required. Therefore, it is strongly recommended that drying shrinkage testing be performed to determine the optimum dosage for each application and each set of materials.

The typical dosage range of Tetraguard AS20 admixture is 0.5 to 1.5 gal/yd^3 (2.5 to 7.5 L/m^3). However, dosages outside of this range may be required depending on the level of shrinkage reduction needed.

Master Builders

Product Data: TETRAGUARD® AS20

Mixing: Tetraguard AS20 admixture may be added to the concrete mixture during the initial batch sequence or at the jobsite.

The mix water content should be reduced to account for the quantity of Tetraguard AS20 admixture used.

If the delayed addition method is used, mixing at high speed for 3-5 minutes after the addition of Tetraguard AS20 admixture will result in mixture uniformity.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: Tetraguard AS20 admixture will neither initiate nor promote corrosion of reinforcing steel, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of Tetraguard AS20 admixture.

Compatibility: Tetraguard AS20 admixture is compatible with all water-reducers, mid-range water-reducers, high-range water-reducers, set retarders, accelerators, silica fume, and corrosion inhibitors. For air-entrained concrete applications, Micro-Air® admixture is the recommended air-entrainer. The dosage of Micro-Air admixture should be established through truck trial evaluations. The trials should include a simulated haul time of at least 20 minutes to assess air content stability. Tetraguard AS20 admixture should be added separately to the concrete mixture to ensure desired results.

Storage and Handling

Storage Temperature: Tetraguard AS20 admixture is a potentially combustible material with a flash point of 208 °F (98 °C). This is substantially above the upper limit of 140 °F (60 °C) for classification as a flammable material, and above the limit of 200 °F (93 °C) where DOT requirements would classify this as a combustible material. Nonetheless, this product must be treated with care and protected from excessive heat, open flame or sparks. For more information refer to the MSDS.

Tetraguard AS20 admixture should be stored at ambient temperatures above 35 °F (2 °C), and precautions should be taken to protect the admixture from freezing. If Tetraguard AS20 admixture freezes, thaw and reconstitute by mild mechanical agitation. ***Do not use pressurized air for agitation.***

Shelf Life: Tetraguard AS20 admixture has a minimum shelf life of 12 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of Tetraguard AS20 admixture has been exceeded.

Packaging

Tetraguard AS20 admixture is available in 55 gal (208 L) drums and 268 gal (1014 L) totes.

Related Documents

Material Safety Data Sheets: Tetraguard AS20 admixture.

Additional Information

For additional information on Tetraguard AS20 admixture or its use in developing concrete mixtures with special performance characteristics, contact your local sales representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative admixtures for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the North American region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

BASF Construction Chemicals
Admixture Systems
www.masterbuilders.com
United States 23700 Chagrin Boulevard, Cleveland, Ohio 44122-5544 ■ Tel: 800 628-9990 ■ Fax: 216 839-8821
Canada 1800 Clark Boulevard, Brampton, Ontario L6T 4M7 ■ Tel: 800 387-5862 ■ Fax: 905 792-0651

BASF
The Chemical Company

3 | 03 30 00 | 03 40 00 | 03 70 00
Product Data
Cast-in-Place Concrete
Precast Concrete
Mass Concrete

Description

Pozzolith 100 XR is a ready-to-use liquid admixture for producing more uniform and predictable quality concrete. Placing and finishing requirements are facilitated because this admixture retards setting time. Pozzolith 100 XR admixture meets ASTM C 494/ C 494M requirements for Type B, retarding, and Type D, water-reducing and retarding, admixtures.

Applications

Recommended for use in:

- Prestressed concrete
- Precast concrete
- Reinforced concrete
- Shotcrete
- Lightweight or normal weight concrete
- Pumped concrete
- 4x4™ Concrete
- Pervious Concrete
- Rheodynamic® Self-Consolidating Concrete

POZZOLITH® 100 XR

Set Retarding Admixture

Features

- Reduced water content required for a given workability
- Retarded setting characteristics
- Controlled retardation – depending on the addition rate
- Dead-load deflection can take place (before concrete sets) in extended pours for bridge decks, cantilevers, nonshored structural elements, etc.

Benefits

- Improved workability
- Reduced segregation
- Superior finishing characteristics for flatwork and cast surfaces
- Flexibility in scheduling of placing and finishing operations
- Offsets effects of early stiffening during extended delays between mixing and placing
- Helps eliminate cold joints
- Peak temperature and/or rate of temperature rise in mass concrete lowered thereby reducing thermal cracking
- Increased compressive and flexural strength

Performance Characteristics

Rate of Hardening: The temperature of the concrete mixture and the ambient temperature (forms, earth, reinforcement, air, etc.) affect the hardening rate of concrete. At higher temperatures, concrete stiffens more rapidly which may cause problems with placing and finishing. Pozzolith 100 XR admixture retards the set of concrete. Within the normal dosage range, it will generally extend the setting time of concrete containing normal portland cement approximately 1-1/2 to 8 hours compared to that of a plain concrete mixture, depending on job materials and temperatures. Trial mixtures should be made with materials approximating job conditions to determine the dosage required.

Compressive Strength: Concrete produced with Pozzolith 100 XR admixture will have rapid strength development after initial set occurs. If retardation is within the normal ASTM C 494/C 494M Types B and D specifications, Pozzolith 100 XR admixture will develop higher early (24-hour) and ultimate strengths than plain concrete when used within the recommended dosage range and under normal, comparable curing conditions.

When Pozzolith 100 XR admixture is used in heat-cured concrete, the length of the preheating period should be increased until initial set of the concrete is achieved. The actual heat-curing period is then reduced accordingly to maintain existing production cycles without sacrificing early or ultimate strengths.

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Product Data: POZZOLITH® 100 XR

Guidelines for Use

Dosage: Pozzolith 100 XR admixture is recommended for use at a dosage of 3 ± 1 fl oz/cwt (195 ± 65 mL/100 kg) of cementitious materials for most concrete mixtures using typical concrete ingredients. Because of variations in job conditions and concrete materials, dosage rates other than the recommended amounts may be required. In such cases, contact your BASF Construction Chemicals representative.

Pozzolith 100 XR admixture may be used at less than the recommended dosage for the purpose of retardation only.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: Pozzolith 100 XR admixture will neither initiate nor promote corrosion of reinforcing steel in concrete. This admixture does not contain intentionally-added calcium chloride or other chloride-based ingredients.

Compatibility: Pozzolith 100 XR admixture may be used in combination with any BASF Construction Chemicals admixtures. When used in conjunction with other admixtures, each admixture must be dispensed separately into the mix.

Storage and Handling

Storage Temperature: If this product freezes, thaw at 35 °F (2 °C) or above and completely reconstitute by mild mechanical agitation. *Do not use pressurized air for agitation.*

Shelf Life: Pozzolith 100 XR admixture has a minimum shelf life of 18 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your BASF Construction Chemicals representative regarding suitability for use and dosage recommendations if the shelf life of Pozzolith 100 XR admixture has been exceeded.

Packaging

Pozzolith 100 XR admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: Pozzolith 100 XR admixture.

Additional Information

For additional information on Pozzolith 100 XR admixture or its use in developing a concrete mix with special performance characteristics, contact your BASF Construction Chemicals representative.

The Admixture Systems business of BASF Construction Chemicals is a leading provider of innovative additives for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets throughout the NAFTA region. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.

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APPENDIX C - DOT SURVEY

1. Crack Resistant Concrete For Use In Bridge Decks

The University of Colorado Denver is conducting a research study funded by the Colorado Department of Transportation. The research involves an investigation into the early-age cracking of bridge decks in Colorado, focusing mainly on concrete mixture designs. The primary goal of this project will be to develop a more crack-resistant concrete for use in bridge decks across Colorado.

The following questionnaire will aid our team in obtaining knowledge that will assist in this study. We greatly appreciate your response to this questionnaire, and any additional comments, suggestions, or concerns that you might have.

If you have any questions, please contact Stephan Durham at the University of Colorado Denver at (303) 352-3894 or by e-mail at stephan.durham@cudenver.edu.

2. Contact Information

* 1. Questionnaire Completed by:

Name:	
Organization:	
Address:	
Address 2:	
Email Address:	
Phone Number:	

3. Bridge Deck Concrete: Cracking Occurrence and Testing

2. Is your state Department of Transportation experiencing bridge deck cracking?

☐ Yes

☐ No

If yes, please include additional information pertaining to any of the bridges and their location, date of construction, type and size of cracks, age when cracking occurred, etc...

3. What do you believe is the primary cause for bridge deck cracking in your state?

- [ ] Placement
- [ ] Curing
- [ ] Rate of Strength Gain
- [ ] Mixture Design
- [ ] Use (or a combination) of Admixtures

Other (please specify)

4. At what age does the concrete being used for bridge decks in your state typically reach its ultimate strength?

- [ ] 3 days
- [ ] 7 days
- [ ] 14 days
- [ ] 21 days
- [ ] 28 days
- [ ] 56 days

5. Does your state Department of Transportation perform laboratory cracking ring resistance testing (AASHTO PP34) on concrete mixture designs used for bridge decks?

- [ ] Yes
- [ ] No

If no, what tests are used to measure the concrete's resistance to shrinkage cracking?

4. Bridge Deck Concrete: Mixture Design

6. As pertaining to mixture design, what do you consider to be the primary cause(s) of your state Department of Transportation's problem with bridge deck cracking?

- [ ] Water to Cementitious Ratio
- [ ] Cement Content
- [ ] Chemical Admixtures
- [ ] Pozzolans

7. What adjustments, if any, have been made to your state Department of Transportation's bridge deck concrete mixture designs or specifications that have resulted in improved concrete bridge deck mixtures?

8. Has your state Department of Transportation placed bridge deck concrete that utilized shrinkage-reducing admixtures (S.R.A.)?

☐ Yes
☐ No

If yes, when were these decks placed and was the use of a S.R.A. beneficial in reducing bridge deck cracking?

9. Has your state Department of Transportation placed bridge deck concrete that utilized shrinkage-compensating cement (SCC)?

☐ Yes
☐ No

10. Which of the materials below, when included in the bridge deck concrete mixture design, increased cracking of your state Department of Transportation's bridge decks?

	Increases	Does Not Influence	Decreases
Silica fume	○	○	○
Class C Fly Ash	○	○	○
Class F Fly Ash	○	○	○
Blast Furnace Slag	○	○	○
Water Reducing Admixtures	○	○	○
Set Retarders	○	○	○
Shrinkage Reducing Admixtures	○	○	○

Please provide any additional information regarding the materials listed above or materials not listed that have increased bridge deck cracking in your state.

11. Which of the materials below, when included in the bridge deck concrete mixture design, proved to be beneficial in reducing cracking of your state Department of Transportation's bridge decks?

	Reduces	Does Not Influence	Increases
Silica Fume	○	○	○
Class C Fly Ash	○	○	○
Class F Fly Ash	○	○	○
Blast Furnace Slag	○	○	○
Water Reducing Admixtures	○	○	○
Set Retarders	○	○	○
Shrinkage Reducing Admixtures	○	○	○

Please provide any additional information regarding the materials listed above or materials not listed that have proved to be beneficial in reducing bridge deck cracking in your state.

12. What is the maximum water to cementitous material ratio allowed for concrete mixtures used for bridge decks in your state?

☐ > 0.45
☐ 0.40 < w/cm < 0.45
☐ 0.35 < w/cm < 0.40
☐ < 0.35

13. Have changes been made to your state Department of Transportation's curing practices which have helped to reduce bridge deck cracking?

☐ Yes
☐ No

If yes, please specify what changes have been implemented.

State	Department Contact	Department Contact
Alabama	conwayf@dot.state.al.us	
Alaska	richard_pratt@dot.state.ak.us	
Arizona	PIO@az511.com	
Arkansas	info@ArkansasHighways.com	
California	richard_land@dot.ca.gov	
Colorado	paulette.shaver@dot.state.co.us	tim.aschenbrener@dot.state.co.us
Connecticut	webmaster.conndot@po.state.ct.us	christine.rjohnson@po.state.ct.us
	robert.rabla@po.state.ct.us	
Delaware	dot-public-relations@state.de.us	
Florida	fdotpio@dot.state.fl.us	william.nickas@dot.state.fl.us
Georgia	david.graham@dot.state.ga.us	rick.deaver@dot.state.ga.us
	paul.liles@dot.state.ga.us	
Idaho	Mike.Ebright@itd.idaho.gov	mfarrar@itd.state.id.us
Iowa	Norman.McDonald@dot.iowa.gov	
Kansas	loren@ksdot.org	
Kentucky	kytc.statehighwayengineer@ky.gov	jimr.king@mail.state.ky.us
Louisiana	BillTemple@dotd.louisiana.gov	hghara@dotd.louisiana.gov
Maine	exec.mainedot@maine.gov	
Maryland	ksaab@mdot.state.md.us	gvaughan@sha.state.md.us
Massachusetts	feedback@mhd.state.ma.us	
Michigan	juntunend@michigan.gov	juntuned@michigan.gov
Minnesota	info@dot.state.mn.us	dan.dorgan@dot.state.mn.us
Mississippi	comments@mdot.state.ms.us	
Missouri	ghanshyam.gupta@modot.mo.gov	chclint@mail.modot.state.mo.us
Montana	kbarnes@state.mt.us	
Nebraska	lfreeman@dor.state.ne.us	
Nevada	info@dot.state.nv.us	wcrawford@dot.state.nv.us
New Hampshire	webmaster@dot.state.nh.us	mrichardson@dot.state.nh.us
	cwaszczuk@dot.state.nh.us	
New Mexico	webmasterHTD@state.nm.us	Sherman.Peterson@nmshtd.state.nm.us
New York	salampalli@gw.dot.state.ny.us	
North Carolina	gperfetti@dot.state.nc.us	
North Dakota	dot@state.nd.us	tudland@state.nd.us
Ohio	john.randall@dot.state.oh.us	odotinfo@odot.org
	Tim.Keller@dot.state.oh.us	Jawdat.Siddiqi@dot.state.oh.us
Oregon	Frank.J.Nelson@state.or.us	
Pennsylvania	penndot_webmaster@state.pa.us	jchiste@state.pa.us
Rhode Island	efparker@dot.state.ri.us	
South Carolina	shealyse@scdot.org	
South Dakota	kevin.goeden@state.sd.us	
Tennessee	TDOT.Comments@state.tn.us	
Utah	bwheeler@utah.gov	
Vermont	Mike.Hedges@state.vt.us	
Virginia	George.Clendenin@VDOT.Virginia.gov	
West Virginia	pmattox@dot.state.wv.us	
Wisconsin	bridge_support@wsdot.wa.gov	webmaster@dot.state.wi.us

APPENDIX D - PHOTOGRAPHS OF CRACKED RESTRAINED RING SHRINKAGE TEST SPECIMENS

Mixture #1 (0.38-6.8-FA20-SF5-II) did not exhibit surface cracking

Mixture #7 (0.44/6.5/BFS50/II) cracked at 32 days of age, and an average of approximately 90micro strain (No Surface Cracking)

Mixture #8 (0.44-6.0-FA30-SRA-II) had not cracked at 56 days of age, and an average of approximately 73micro strain (No Surface cracking)

Mixture #3 (0.38-6.8-FA20-SF5-G) Ring1

Mixture #2 (0.42/6.2/FA16/SF3.5/II), Ring 2

Mixture #4 0.42/6.2/FA16/SF3.5/G), Ring 2

Mixture #5 (0.44/6.5/FA30/II), Ring #1

Mixture #6 (0.44/6.5/FA30/SF5/II), Ring #2

Mixture #9 (0.44-6.0-FA30-RET-II), Ring #1

Mixture #10 (0.42-6.0-II-L.W.A)

Mixture #11 (0.42-6.0-II-Norm.Wt.)