EVALUATION OF LIGHTWEIGHT CONCRETE MIXTURES FOR BRIDGE DECK AND PRESTRESSED BRIDGE GIRDER APPLICATIONS

by

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B.S., Kansas State University, 2006

A THESIS

submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering
College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

2008

Approved by:

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Abstract

As of 2005, 23% of the bridges in the Kansas infrastructure are classified as structurally deficient or functionally obsolete according to the ASCE Infrastructure Report Card (ASCE, 2008). One alternative to replacing the entire bridge structure is replacing only the superstructure with lightweight concrete. This option is more economical for city, county, and state governments alike. Replacing the superstructure with lightweight concrete can oftentimes allow the bridge rating to be upgraded to higher load capacities or higher traffic volumes. Furthermore, lightweight concrete can be used initially in a bridge deck to provide reduced weight and a lower modulus of elasticity, therefore lower cracking potential.

The Kansas Department of Transportation is interested in the potential benefits of using lightweight aggregate concrete in Kansas bridge decks and prestressed bridge girders. This research project used three types of lightweight aggregate to develop lightweight concrete mixtures for a bridge deck and for prestressed bridge girders. Two of the lightweight aggregates were expanded shale obtained locally from the Buildex Company. One deposit was located in Marquette, Kansas, and the other in New Market, Missouri. The third lightweight aggregate source was expanded slate obtained from the Stalite Company in North Carolina. Aggregate properties including absorption, gradation, and L.A. Abrasion were evaluated.

Over 150 lightweight concrete mixtures were created and tested and several mix design variables such as water-to-cement ratio, cement content, and coarse-to-fine aggregate ratio were evaluated. From these results, optimized bridge deck and optimized prestressed concrete mixtures were developed for each type of lightweight aggregate. Special concerns for lightweight aggregate concrete are addressed.

These optimized concrete mixtures were then tested for KDOT acceptability standards for the concrete properties of compressive strength, tensile strength, modulus of elasticity, freeze-thaw resistance, permeability, alkali-silica reactivity, drying shrinkage, and autogenous shrinkage. All concrete mixtures performed satisfactorily according to

KDOT standards. In addition, an internal curing effect due to the moisture content of the lightweight aggregate was observed during the autogenous shrinkage test.

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Acknowledgements

This project required the effort, time, knowledge and skill of several people. First, I would like to express sincere gratitude to Dr. Robert J. Peterman for his supervision and insight throughout this project. Next, I would like to thank Mr. David Meggers of KDOT for his guidance and unending support, and Dr. Brian Coon for his instruction and assistance. I would also like to thank Dr. Asad Esmaeily for serving on my committee. In addition, I would like to credit Mr. Jake Perkins, fellow graduate student and research partner, for his patience, continued effort and assistance, and positive attitude.

I would also like to extend thanks to the Federal Highway Administration and the Kansas Department of Transportation for funding this research project. In addition, I would like to thank the KDOT technicians (especially Jim Bernica) for all of their time and assistance with several of the concrete tests. Furthermore, I would like to acknowledge my fellow graduate students that helped throughout the project, especially Patrick Sheedy, and all of the other Civil Engineering faculty and staff (especially Peggy Selvidge, Danita Deters, and Dave Shuling), and undergraduate students (especially Lisa Beck, Logan and Ryan Benteman, Brady Hedstrom, and Jordan Dettmer) that have aided in this project.

In addition, I would like to thank my parents, David and Pennie Grotheer, for their continued encouragement and support. Finally, I would like to express my appreciation to Dr. Kyle Larson for all of his assistance and support throughout this project.

CHAPTER 1 - Introduction

Overview

As of 2005, 23% of the bridges in the Kansas infrastructure are classified as structurally deficient or functionally obsolete according to the ASCE Infrastructure Report Card (ASCE, 2008). These bridges were initially designed to carry lighter traffic loads or volumes compared to the present-day design loads, thus obtaining their current classifications. Replacing the entire structure is very expensive and time consuming. However, one potential alternative to this is replacing only the superstructure with lightweight concrete. This option is more economical for city, county, and state governments alike. Furthermore, replacing the superstructure with lightweight concrete can oftentimes allow the bridge rating to be upgraded to higher load capacities or higher traffic volumes if extra lanes are added. Furthermore, lightweight concrete can be used initially in a bridge deck to provide reduced weight and a lower modulus of elasticity, therefore lower cracking potential.

Lightweight structural concrete, also known as high-performance lightweight concrete, has several desirable and beneficial characteristics such as lower modulus of elasticity, improved microstructure, internal curing, and reduced dead load. Lightweight structural concrete is defined by ACI 213 as concrete with an air-dry density in the range of 85 to 115 pcf, with some job specifications allowing air-dry densities up to 120 pcf, and a 28-day compressive strength greater than 2500 psi (ACI Committee 213, 1999). This is a significantly reduced dead load compared to normal-weight concrete with an air-dry density of about 140 pcf. The reduction in density is achieved by using lightweight aggregate, usually composed of expanded shale or slate. Furthermore, when lightweight concrete is used for bridges, several potential benefits emerge including increased width or number of traffic lanes, increased load capacity, balanced cantilever construction, reduction in seismic inertial forces, increased cover with equal weight, improved deck geometry, and longer spans saving pier costs (ACI Committee 213, 1999).

Lightweight aggregate is the primary difference between normal-weight and lightweight concrete. There are several types of lightweight aggregates including Vermiculite, Perlite, pumice, scoria, expanded shale, expanded clay, expanded slate, fly ash, and slag. Typically, an inverse correlation exists between density of the aggregate and compressive strength of the concrete when all other variables are constant. Lower density aggregates are primarily used for insulating or moderate-strength concrete applications. Higher density aggregates, such as expanded shales, clays, slates, slags, pumice, and scoria, which yield higher strength concrete, are used for structural lightweight concrete applications (ACI Committee 213, 1999). The process of expanding lightweight aggregate in a kiln was developed by Stephen Hayde in the early 1900s in Kansas City, Missouri (Buildex Inc., 2006). The first major project to use lightweight concrete was in World War I when the American Emergency Fleet Corporation built lightweight concrete ships from 1917 to 1920 (American Concrete Institute, 2006). "The entire hull structure of the USS Selma and 18 other concrete ships were constructed with 5000 psi, high-performance lightweight concrete in the ship building program in Mobile, Alabama, starting in 1917" (American Concrete Institute, 2006). Since then, structural lightweight concrete has been used for countless purposes and structures including precast structures, high-rise buildings, and bridges. Two well-known lightweight bridges include the San Francisco-Oakland Bay Bridge and the replacement of the Tacoma Narrows Bridge, which incorporated additional traffic lanes due to reduced dead load (ACI Committee 213, 1999).

This research program was initiated by KDOT to investigate potential benefits of using lightweight concrete for a bridge deck and prestressed bridge girders. More specifically, this portion of the project was designed to evaluate material properties of lightweight aggregate and potential lightweight concrete mix designs.

Objectives

There were three main phases of this research project. The main objective of the first phase was to gather information on the background and current uses of lightweight concrete and any recent research being conducted. The second phase consisted of creating and testing several preliminary lightweight concrete mix designs. Finally, the third phase evaluated several concrete properties of optimized lightweight concrete mix designs created during phase two.

To gather information and background on current uses and practices regarding lightweight concrete, industry standards and practices such as ACI 213 and the PCA Design and Control of Concrete Mixtures were reviewed. Information was also obtained on the material properties and availability of lightweight aggregate. In addition, previous lightweight concrete mix designs were obtained from KDOT, along with a history of existing lightweight bridges in the area. Finally, current KDOT mix design specifications were attained and reviewed.

The second phase of this research project was to create and test several lightweight concrete mix designs. Three different types of lightweight aggregate were obtained and used to develop lightweight concrete mixtures. Two sources were local to Kansas, coming from shale deposits located in Marquette, Kansas, and New Market, Missouri, both owned by the Buildex Corporation. The third aggregate source was expanded slate obtained from the Stalite Corporation in North Carolina. Stalite was chosen for its high-quality reputation, to be used as a baseline aggregate to compare the results of the local Kansas lightweight aggregate. Lightweight mix designs were then created and tested. Several mix design variables were altered, such as water-to-cement ratio, cement content, and coarse-to-fine aggregate ratio, in order to find an "optimized" mix design. For this research project, an "optimized" mix design refers to the combination of mix design variables that produces concrete with the required compressive strength, slump, workability, and unit weight desired, while also keeping factors such as economy, shrinkage potential, and air content at reasonable levels. First, an optimized mix design was achieved for a potential bridge deck, followed by an optimized mix design for a prestressed bridge girder. This mix design process was complicated by the fact that the lightweight aggregate can have up to 30% absorption. For this reason, the mix design and concrete batching process must be altered from that of normal-weight concrete. For example, lightweight aggregate must be pre-soaked prior to batching. Therefore, the corresponding moisture content of the lightweight aggregate is greatly altered and the water-to-cement ratio of the concrete mix design must be corrected. Pre-soaking aggregate concerns were addressed both for small-scale lab batches and large-scale production batches in this phase.

Finally, the third phase of this project was to perform comprehensive material property tests on the "optimized" mix designs for both the bridge deck and prestressed girders for each of the three lightweight aggregate types. Material property tests included absorption, gradation, and L.A. Abrasion for the lightweight aggregate. Material property tests for the lightweight concrete included compressive strength, tensile strength, modulus of elasticity, freeze-thaw durability, permeability, alkali-silica reactivity, drying shrinkage, autogenous shrinkage, and coefficient of thermal expansion.

Scope

Chapter Two reviews the current literature and research that has been published on lightweight concrete. Topics specifically focused on were lightweight concrete being used by state department of transportations and material property tests such as drying shrinkage and autogenous shrinkage. In addition, guidelines from industry standards such as ACI 213 and the PCA Design and Control of Concrete Mixtures are reviewed.

Chapter Three discusses the background behind all three lightweight aggregate sources used in this research project. Material properties of each aggregate, such as gradation, density, absorption, and L.A. Abrasion are reported. Benefits and drawbacks of each aggregate type are also assessed.

Chapter Four reviews the concrete mixture proportioning and design process. Concrete mix design goals are discussed for both the bridge deck and prestressed concrete mixtures. Mix design variables are reviewed. Preliminary concrete mixture results are given. Finally, optimized bridge deck and prestressed concrete mixtures are shown.

Chapter Five reviews all experimental test setups for all of the concrete property tests including compressive strength, tensile strength, modulus of elasticity, freeze-thaw resistance, permeability, alkali-silica reactivity, coefficient of thermal expansion, and shrinkage.

Chapter Six covers all concrete property test results for both the optimized bridge deck and optimized prestressed concrete mixtures. In addition, acceptability for KDOT bridge deck and prestressed bridge beams is discussed.

Chapter Seven gives overall conclusions and recommendations from the project.

CHAPTER 2 - Literature Review

This chapter contains a review and summary of published articles and industry manuals relevant to this research project. Works reviewed include topics of performance of existing lightweight structures, industry standards for lightweight concrete, material properties, and drying and autogenous shrinkage.

Industry Standards

Throughout this project, several sources have been used to determine industry standards and practices of lightweight concrete. Some of these sources of particular importance to the research conducted on lightweight concrete are discussed here.

The Expanded Shale, Clay, and Slate Institute (1971) published material on the history, applications, and economics of lightweight concrete. This work describes how expanded shale aggregate was discovered by Stephen J. Hayde of Kansas City, Missouri, in 1908. The first large-scale production of lightweight concrete was used in World War I by the United States Fleet Corporation to build ships, including the U.S.S. Selma. Aggregate properties and the production process are described in this manual. The work goes on to list several bridges and buildings that have been constructed with lightweight concrete. Some of the well-known projects discussed are San Francisco-Oakland Bay Bridge, Chicago's Lake Point Tower, and the Los Angeles Dodgers Stadium. In several instances, use of lightweight concrete was chosen due to design constraints and economy. Examples and details of cost savings for several projects are also included.

The Federal Highway Administration has published *Criteria for Designing Lightweight Concrete Bridges* (1985). This document covers several aspects of lightweight concrete including history; lightweight aggregates and their production; a synopsis of major lightweight bridges in the U.S., Europe, and Canada; a survey of 30 existing lightweight bridges in the U.S. and an in-depth report on the performance of 12 of them; and an economic analysis of several types of bridges using lightweight concrete. Overall, the survey of existing bridges showed satisfactory performance results. A few state departments of transportation reported problems with lightweight concrete, but most of these issues can be attributed to improper mix design, poor quality control at time of placement, and inexperienced construction methods. Several examples are given of acceptable lightweight bridge performance for many decades. In addition, economic benefits are demonstrated by bridges that were able to be improved by redecking with lightweight concrete to increase the number of traffic lanes or upgrade load capacity. Finally, recommendations are given and the need for good quality control during batching and placement is emphasized.

ACI Committee 213 produced the *Guide for Structural Lightweight Aggregate Concrete* (1999). This is a comprehensive report designed to give material background for lightweight aggregate and lightweight concrete, and offer in-depth information on mix design, testing, and placing. The list of subjects covered includes history, economy, lightweight aggregate structure, properties and production, proper proportioning, placement techniques, and several physical and mechanical properties of structural lightweight concrete.

Performance of Existing Lightweight Structures

Using lightweight concrete can be beneficial, both structurally and economically. Several state departments of transportation (DOT) have taken advantage of these benefits and utilized lightweight concrete in both bridges and roads. The following article summaries demonstrate how lightweight concrete has performed in other DOT projects.

Brown et al. (1995) evaluated the long-term performance of structural lightweight concrete in a four-span precast, prestressed bridge framing system and cast-in-place bridge deck slab constructed in Florida. An in-depth investigation was conducted in 1968 and again in 1992 to determine service load strains and deflections, and then compared these results to predicted theoretical bridge responses. The evaluation showed that both measured deflections and strain measurements indicated no increase in flexibility over time. This study also reviewed the surface-wearing characteristics of another lightweight concrete Florida bridge after 30 years of exposure. On this bridge, lightweight structural concrete was used on part of the bridge and normal-weight concrete on the other portion, separated by an expansion joint. This side-by-side comparison revealed that wear of the structural lightweight concrete was essentially the same, if not slightly better, than the normal-weight concrete. In both bridges, structural lightweight concrete met and exceeded the Florida Department of Transportation's expectations.

Stolldorf and Holm (1996) reported that the elevated section of the Whitehurst Freeway in Washington, D.C., was successfully upgraded from an H20 live load to an HS20 loading criteria by using structural lightweight concrete to replace the existing normal-weight deck. The upgrade required widening the existing deck by eight feet in most places. Initial rehabilitation studies for this project focused on strengthening and replacing the deck with normal-weight concrete. However, this option proved to be unfeasible with both the higher HS20 live load and the additional lane width. This article also stated that lightweight concrete bridge decks have been widely used in the District of Columbia, Maryland, and Northern Virginia for more than 40 years. In addition, observations of the superior wearing characteristics of mature, exposed lightweight concrete decks have led the common practice in these areas to be leaving the lightweight deck uncovered.

Ozyildirim and Gomez (2005) reported on the first high-performance lightweight concrete bridge constructed for the Virginia Department of Transportation. The project included a performance study of lightweight concrete, implementing the concrete into prestressed beams and a bridge deck, conducting condition surveys of the bridge for four years, and an estimation of benefits and costs associated with using lightweight concrete. In this study, extensive material property tests were conducted including slump, unit weight, temperature, air content, compressive strength, flexural strength, permeability, elastic modulus, freeze-thaw resistance, and drying shrinkage. In addition, prestressed beams were tested to measure transfer length, development length, and flexural strength. Most of these tests were considered to pass satisfactorily, except for one of the test batches where excess water was suspected to have been in the mix. In this case, the lightweight aggregate was pre-wetted. However, it is believed that moisture content throughout the aggregate stockpile was not consistent, allowing excess water to be unaccounted for in the concrete. The main conclusion of this report is that high-performance lightweight concrete can be proportioned and produced to be lightweight, workable, strong, volumetrically stable, and durable. The beams evaluated had shorter transfer lengths than were predicted. Better control of excess water from aggregate moisture is required when batching lightweight concrete. The volumetric air content can delay placement; therefore, inspectors should use density measurements to control the air content once a relationship is established. Furthermore, the condition survey of the bridge after four years indicated limited cracking and no increase in cracking from previous inspections. Finally, the Virginia DOT expects that the higher initial cost of lightweight concrete will be offset by enhanced durability and extended service life, resulting in a reduction in life-cycle costs of at least 10 percent.

Raithby and Lydon (1981) reported on lightweight concrete highway bridges used around the world. The article highlighted lightweight bridges in America, since lightweight aggregate was first made in the U.S. and lightweight structures have, therefore, been in use longer. American structures discussed included the San Francisco-Oakland Bay Bridge, redecking the Golden Gate Bridge, 1950 reconstruction of the Tacoma Narrows Bridge, and the Chesapeake Bay Bridge. The article also pointed out several major European bridges completely or partially composed of lightweight concrete. Construction and durability issues of these projects were discussed, but in most cases it was found that problems can be avoided with proper mixture proportions and proper quality control. In addition, economic benefits of several different projects utilizing lightweight concrete were given.

Material Properties

Material properties of lightweight aggregate and lightweight concrete were the main focus of this research project. An extensive effort was made to review several articles and research that evaluated these subjects and to gain an understanding of current knowledge and practices in the subject.

Gray et al. (1961) studied fatigue properties of two lightweight aggregate concrete mixtures using S-N relationships and further compared these results to previously established S-N curves for normal-weight concrete. The testing program consisted of a low-strength and a high-strength lightweight aggregate concrete mixture. Each series was composed of five batches of 30 cylinders each. The cylinders were loaded in direct compression at stress levels of 40, 50, 60, 70, and 80 percent of the estimated ultimate strength for 10 million cycles. The main conclusion of this study was that fatigue properties of lightweight concrete are not different over large variations of strength, nor do they significantly differ from fatigue properties of normal-weight concrete.

Hoff (1994) reviewed fatigue behavior of high-strength lightweight concrete tested under a variety of conditions. The study primarily focused on fatigue behavior, both compressive and flexural, for concrete in a marine environment. Results of several studies showed that high-strength lightweight concrete performs at least as well as high-strength, normal-weight concrete, and in many instances even better. The primary reason believed to allow the lightweight concrete to perform better is attributed to the lower modulus of the lightweight aggregate and the improved interfacial zone between the lightweight aggregate and the paste matrix. These features lead to reduced microcracking within the concrete, thus improving fatigue behavior.

Holm and Bremner (1984) report on the long-term durability of structural lightweight concrete. They reviewed the unique characteristics of lightweight concrete that increase its durability, namely expanded aggregate properties. Expanded clay, shale, and slate aggregates contain non-interconnected pores, can be pozzolanic, and have a similar stiffness to the surrounding paste matrix. All of these characteristics help to increase durability. Laboratory rapid-freezing and thawing tests were conducted and an attempt was made to correlate this data to that of existing lightweight structures exposed to severe weather conditions. Structures analyzed included ships and marine structures, bridges, and industrial structures. The overall conclusion was that properly proportioned, structural lightweight concrete was shown to perform well in various applications where severe exposure conditions exist.

Bremner and Holm (1986) also reported on elastic compatibility and behavior of concrete. Since concrete is a composite material, interaction between the constituents of the composite play an important role in how the material will behave. A method of stress analysis developed by Goodier was used to quantify the potential stress concentration reduction between normal-weight and lightweight aggregate. In normal-weight concrete, the elastic modulus of the aggregate is considerably higher than that of the surrounding matrix. This effect is exaggerated even more if the concrete is air-entrained. However, porous lightweight aggregate has an elastic modulus similar to the paste matrix, and is usually even more closely matched to the matrix of air-entrained concrete. The result is a more continuous phase within the composite material, allowing for better bond within the contact zone and less development of stress concentrations.

Bremner et al. (1984) studied aggregate-matrix interaction in concrete subjected to severe weathering conditions. The contact zone between the aggregate and mortar matrix was given particular attention, noting that failure can come from the aggregate, mortar, or contact zone connecting the two. Therefore, good bond within the contact zone is required for high-quality durability. This study used a scanning electron microscope equipped with an energy-dispersive X-ray analyzer to determine the contact zone bond. Several structural lightweight concrete specimens from existing structures exposed to severe conditions were examined and compared to normal-weight concrete specimens. The study indicated that lightweight concrete developed sufficient bond between the expanded aggregate and mortar matrix. However, normal-weight concrete showed signs of cracking in the contact zone.

Holm et al. (1984) reported on the effects of long-term exposure of lightweight concrete subjected to severe weathering conditions. The contact zone between the lightweight aggregate and mortar matrix was analyzed, and results showed no evidence of debonding or microcracking. Reasons for these impressive findings were investigated, and it was found that the aggregate-mortar interaction is mechanical and chemical. A pozzolanic reaction exists between the expanded aggregate and the cement hydration products, in addition to the mechanical bond between the two. Also, elastic compatibility of the lightweight aggregate and mortar matrix are much more similar than that of normal-weight aggregate and mortar. This similarity in stiffness reduces stress concentration at the aggregate mortar interface, thus reducing the potential for microcracking. Finally, another reason for reduced microcracking is the ability of the porous lightweight aggregate to absorb bleed water during the early phase of the hydration process, then subsequently release the water for an extended period of internal curing, resulting in a higher-quality matrix.

Fujji et al. (1998) did an extensive study on the properties of high-strength and high-fluidity lightweight concrete. Lightweight concrete mixes with high fluidity and workability, and with compressive strengths of 8700 psi, were targeted. Silica fume-blended cement and belite-rich cement were used. Effects of water-to-cement ratio, curing temperature, curing method, and type of water-reducing admixture were studied. Tests conducted included flow and flow time also known as spread, compressive strength, elastic modulus, air permeability, water permeability, total pore volume, and amount of $Ca(OH)_2$ formation.

Zhang and Gjorv (1991) researched the permeability of high-strength lightweight concrete compared to that of normal-weight concrete. In harsh environments, permeability largely controls durability of the concrete. This study examined both water penetration and chloride penetration for lightweight and sand-lightweight mixes. The article noted that other studies have observed equal or lower permeability of lightweight concrete compared to normal-weight concrete, and attributed the result to improved interfacial zone, a more unified structure, and a reduction of internal stress due to volume changes in the initial unloaded states. Results of this study found that overall permeability of the high-strength lightweight concrete was low, but appeared to be more dependent on the porosity of the matrix rather than the porosity of the aggregate. An optimum cement content was also observed where too much cement actually increased the permeability. Finally, accelerated chloride penetration was a good indication of permeability, but electric conductivity showed no relationship.

Bremner et al. (2007) evaluated the influence of expanded-shale lightweight aggregate in concrete with reactive-alkali aggregate. In the study, cements with high silica content were also used to further facilitate the undesirable alkali-silica reaction (ASR). Lightweight fine aggregate was used to replace normal-weight sand in varying amounts. Effects of ASR were then evaluated for four years. Results showed that replacement of normal-weight aggregate with lightweight aggregate effectively suppressed ASR production. With 100% replacement of lightweight aggregate, expansion at one year was only a third of that compared to normal-weight aggregate. ASR reduction is believed to be caused by the pozzolanic nature of the expanded shale and the presence of approximately 40% voids within the lightweight aggregate.

Vaysburd (1996) reported on the durability of lightweight concrete in bridges exposed to severe environments. This report first discussed material properties of the lightweight aggregate, compared to normal-weight aggregate, that allow the material to have improved durability performance. These properties included elastic compatibility between lightweight aggregate and the surrounding mortar matrix, and lightweight aggregate pore characteristics. The report described how normal-weight aggregates draw films of bleed water to individual aggregate particles, resulting in decreased bond within the contact zone of the aggregate and cement matrix. In contrast, the porous, lightweight aggregate absorbs any extra bleed water surrounding the aggregate particle. When enough cement hydration has occurred and the relative humidity in the concrete falls below 80 percent, the internally absorbed water within the lightweight aggregate will be drawn out to be used for internal curing. This process, in addition improved elastic compatibility, results in decreased microcracking and improved durability. The author also suggested that when mixing lightweight concrete, the cement mortar should be mixed for a few minutes prior to adding the lightweight aggregate. He has observed that this process, although opposite of the traditional batching sequence, will reduce the amount of water absorbed by the lightweight aggregate from the mix by 30 to 50 percent. An additional benefit of this method is that the aggregate particle absorbs "cement milk," rather than just water, contributing to the formation of a stronger contact zone.

Shrinkage

Zhutovsky et al. (2002) researched the efficiency of using lightweight aggregates for internal curing to eliminate autogenous shrinkage. This study looked at three different sizes of lightweight pumice sand, and focused on the variables of aggregate pore size and spacing between the individual aggregate particles. The amount of water in the internal reservoirs of the aggregate pores was calculated from the chemical shrinkage that occurs. However, several studies have found that additional water is required, above the calculated amount, because not all of the water in the aggregate pores is available for hydration. Concrete specimens with three different sizes of pumice sand and a reference mix were evaluated for autogenous shrinkage. The best performance was shown by the largest size of pumice aggregate, having nearly all of the autogenous shrinkage eliminated. Therefore, results showed that pore size, and not aggregate particle spacing, was the governing variable to decrease autogenous shrinkage.

Takada et al. (1998) evaluated the autogenous shrinkage of concrete, with partial or complete replacement of normal-weight aggregate, with varying degrees of saturated lightweight aggregate. Since self-desiccation and autogenous shrinkage are more likely to cause cracking in high-performance concrete, the mixes considered were high strength, with a 0.37 water-to-cement ratio. Two main test series were investigated in this study. The first consisted of partial replacement of normal-weight aggregate with fully saturated lightweight aggregate in the percentages of 10%, 17.5%, 25%, and 100%. The second series consisted of full replacement of normal-weight aggregate with lightweight aggregate, with 20% and 60% partially saturated lightweight aggregate. Compressive strength was also measured to determine the effect of replacing lightweight aggregate on strength. The study found that use of saturated lightweight aggregates can affect the volume change drastically. Partial replacement of normal-weight aggregate with lightweight aggregate reduced shrinkage by as much as half at 144 hours for the 25% replacement. Furthermore, all lightweight mixes with 60% and 100% saturation caused expansion, completely overcoming and eliminating any autogenous shrinkage from occurring. Noticeable strength reduction was not seen for lightweight aggregate replacement up to 25%, and strength reduction was small for aggregate replacement over 25%.

Nassif et al. (2003) looked at early-age and drying shrinkage and the effect of curing methods on high-performance concrete. Autogenous shrinkage is also evaluated since with the low water to cement ratios typically used in high-performance concrete, autogenous shrinkage can cause cracking. The three different curing conditions consisted of air-drying, burlap or moist curing, and the use of a curing compound. Concrete mixtures containing fly ash, silica fume, and slag were evaluated with normal-weight coarse aggregate, and both lightweight and normal-weight fine aggregate. The lightweight fine aggregate was obtained from Norlite Co. Admixtures consisted of a W.R. Grace superplasticizer, DARACEM-19, and an air-entraining agent, DARAVAIR-1000. All concrete mixtures were at water-to-cementitious ratios of either 0.29 or 0.35. All specimens were cast and cured in an environmental chamber under constant temperature of 77°F, and relative humidity of 50%. To measure the shrinkage for each concrete mixture, nine 3 in. x 3 in. x 11 in. prisms were used, cast with 2 in. vibrating wire strain gages (VWSGs). The VWSGs were connected to a data logger and readings were taken at five minute intervals for one week. After one week, readings were taken daily for 28 days, and then recorded on a weekly basis. In addition, steel studs were embedded into the ends of the prisms so that drying shrinkage could be measured with a length comparator according to ASTM C157. Results showed that the highest shrinkage occurred for the air-dry cured specimens. The concrete mixtures containing fly ash performed better than other mixtures showing that the addition of fly ash can improve performance by slowing down the rate of hydration. Shrinkage was also reduced in the concrete mixtures with higher water-to-cementitious ratios, since the water contained in these high-performance mixes was quite scarce and at the low 0.29 water-to-cementitious ratio self-desiccation likely caused greater autogenous shrinkage. Results also showed that the concrete mixtures containing lightweight aggregate have less shrinkage than those with normal-weight aggregate. This affect is attributed to the lightweight aggregate having a higher moisture content and supplying additional water for cement hydration to occur, producing an internal curing affect and making the specimens less susceptible to shrinkage.

Duran-Herrera et al. (2007) studied the effect of a 20% substitution of normal-weight fine aggregate with saturated fine lightweight aggregate on the development of autogenous shrinkage of a 0.35 water-to-binder ratio high-performance concrete. A control concrete containing all normal-weight fine aggregate was used for comparison. Shrinkage was monitored using vibrating wire gages located at the center of 4 in. x 4 in. x 16 in. concrete specimens. The autogenous shrinkage specimens were sealed with self-adhesive aluminum tape to create a closed-curing environment. The drying shrinkage specimens were cured under water for six days, then removed from the water and maintained at 73°F and 50% relative humidity. Results showed that the specimens made with 20% fine lightweight aggregate swelled more, and for a longer time, than the reference concrete. The substitution of 20% lightweight fine aggregate for normal-weight fine aggregate led to an 80% reduction in autogenous shrinkage compared to the reference concrete. In addition, the 20% substitution of lightweight aggregate led to a reduction in drying shrinkage of 40, 30, and 20% at the ages of 7, 28, and 91 days, respectively. At 91 days, autogenous shrinkage was 25% of the drying shrinkage for the 20% lightweight substitution mix, and 60% for the reference concrete. From the results, conclusions can be made that a 20% replacement of normal-weight fine aggregate with saturated lightweight fine aggregate can significantly reduce autogenous shrinkage, and this substitution does not significantly affect the concrete unit weight, compressive strength, or chloride-ion permeability.

Kohno et al. (1999) looked at the effects of lightweight aggregate on autogenous shrinkage of concrete. In this study, three types of aggregate evaluated were expanded shale lightweight aggregate, crushed stone normal-weight aggregate, and an artificial lightweight aggregate made from palletized and coated finely ground perlite powder. The factors influencing autogenous shrinkage were the type, moisture content, and unit quantity of aggregate. Moisture contents of absolutely dry, immersed for 24 hours, and boiled for two hours were evaluated. For the first 24 hours, length change was measured in the mold and specimens were wrapped in polyester film. After demolding, specimens were sealed by adhesive aluminum tape to prevent water evaporation. Autogenous shrinkage was measured by a 4-in. strain gage and thermocouple embedded in the specimen. Results showed that specimens containing expanded shale swelled rapidly until the age of one day, then remained stable at about 180×10^{-6}. The artificial lightweight aggregate specimens showed some autogenous shrinkage, and the crushed stone specimens had the largest autogenous shrinkage. Therefore, autogenous shrinkage is reduced by using lightweight aggregate. Furthermore, the moisture content of the aggregate had a significant affect on autogenous shrinkage with autogenous shrinkage decreasing with increasing moisture content. In addition, an increase in unit quantity of lightweight aggregate was found to reduce autogenous shrinkage.

Zhang et al. (2005) evaluated the shrinkage of high-strength lightweight aggregate concrete exposed to a dry environment for two years. In this study two types of expanded clay lightweight aggregates were compared to crushed granite aggregate for 0.34 water-to-cement ratio concrete mixtures of the same batch proportions, and to 0.51 water to cement ratio concrete mixtures containing crushed granite. The 0.51 water-to-cement ratio mixtures were created to achieve the same 28-day compressive strength as the lightweight aggregate mixes. In addition, 5% silica fume was added to some of the mixtures to evaluate its affect on shrinkage. The lightweight coarse aggregate was presoaked for one hour prior to batching. Concrete specimens, 4-in. x 4-in. x 16-in. were cast and covered with wet burlap and a plastic sheet for 24 hours, then demolded.

Shrinkage specimens were cured for seven days in 100% relative humidity moist room, then removed and placed in a conditioned room with a temperature of 86°F and 65% relative humidity where they were monitored for two years. Length change was measured by pins glued on the ends of specimens and a dial gage. Results showed that shrinkage decreased with a decrease in aggregate density that corresponded to an increase in aggregate porosity. This affect is attributed to the moisture content within the lightweight aggregate pores causing an internal curing affect that lowers the autogenous shrinkage of the concrete. In addition, shrinkage was also reduced in the concrete containing silica fume, and the reduction observed was greater for lightweight concrete.

Tazawa and Miyazawa (1997) looked at the effect of cement type, water-to-cementitious ratio, volume concentration of aggregate, and admixtures on autogenous shrinkage. For this study, cement paste, mortar, and concrete specimens were used. To measure the autogenous shrinkage of the concrete, 4-in. x 4-in. x 12-in. specimens were sealed with aluminum tape. Other specimens in the study were subjected to drying or kept under water. The horizontal length change was measured with a dial gage and contact chips attached to two surfaces of the specimens, and by embedded strain gages with low stiffness. Results showed that autogenous shrinkage increases with decreasing water-to-binder ratio. The autogenous shrinkage observed at the age of two months is 100×10^{-6} for a 0.40 water-to-binder ratio and 200×10^{-6} for a 0.30 water-to-binder ratio. These numbers suggest that a significant amount of shrinkage is due to autogenous shrinkage and cracking can be affected. In addition, larger autogenous shrinkage was observed for high-early-strength cement paste and the composition of the cement has a greater influence on autogenous shrinkage than on drying shrinkage. Finally, autogenous shrinkage decreases with increasing aggregate volume.

Lepage et al. (1999) studied early shrinkage development in high-performance concrete. In this study, the Standard Test Method for Length Change of Hardened Hydraulic Cement Mortar and Concrete, ASTM C157, is reviewed. The main drawback of measuring shrinkage by this method is missing the autogenous shrinkage that develops within the first 24 hours after casting. For this reason, the authors evaluate autogenous shrinkage using two types of vibrating wire gages, a more rigid but easy to install 4-in. gage and a lower rigidity, lower cost 2-in. gage. Specimens were cast in plywood forms and kept in place for two days while shrinkage measurements were measured in the molds. It is assumed that the affect of drying shrinkage is not seen in the interior of the concrete specimens where the gages are measuring length change, so the only length change being measured is due to autogenous shrinkage. Results showed that both the 2-in. and 4-in. vibrating wire gages produced similar results. Furthermore, if following procedures of ASTM C157 the autogenous shrinkage developed within the first 24 hours, 259 $\mu\varepsilon$ for this particular experiment, would have been missed. This shrinkage represents 63% of the actual 28-day autogenous shrinkage.

Internal Curing

Barrita et al. (2002) investigated internal curing from saturated lightweight aggregate using magnetic resonance imaging. For this study, 11% saturated lightweight aggregate was incorporated into normal-weight concrete mixes. Two types of specimens were evaluated. The first type was a single, saturated lightweight aggregate placed in contact with a cement mortar mixture to examine moisture transfer from the larger aggregate pores to the smaller matrix pores during cement hydration. The second type was concrete cylinders with saturated lightweight aggregate incorporated into the mixture, cured for varying amounts of time, then dried to determine retained moisture content. Results showed that most of the water transfer from aggregate pores to the surrounding matrix occurred within the first 24 hours, and more than 50% of the moisture originally in the pores was retained by the aggregate and not available for hydration. Overall results from the cylinders indicated that moist curing was still required for all mixes to reduce evaporation of water at an early age.

Villarreal and Crocker (2007) reported on use of partial replacement of normal-weight aggregate with lightweight aggregate for internal hydration. A laboratory study was first conducted to see the effects of varying amounts of lightweight aggregate replacement. Substitutions of 3, 5, and 7 ft^3/yd^3 of lightweight aggregate was used to replace the normal-weight aggregate; and effects of workability, density, and compressive strength were observed. Results showed that the 3- and 5-ft^3/yd^3 substitutions improved compressive strength and workability when compared to standard-mix cylinders, while the 7-ft^3/yd^3 substitution decreased the strength. Based on these results, the 5-ft^3/yd^3 replacement was chosen to be implemented in the field for paving projects. The objective was to reduce shrinkage due to improved curing and minimize shrinkage cracking. About 550,000 yd^3 of paving concrete with internal hydration had been used in the Fort Worth, Texas, area at the time of this report. Reports of common cracks from plastic or drying shrinkage have been low. In addition, internal hydration is believed to be effective since compressive strength and workability results are improved from similar standard paving mixes. Another benefit is the 200-lb. reduction in weight of a cubic yard of concrete, allowing a typical 10-yd^3 load to be increased by 0.5 yd^3 without increasing overall weight, thus saving fuel and equipment wear.

Kovler et al. (2003) looked at using small amounts of lightweight aggregates to replace normal-weight aggregate and still achieve adequate internal curing to mitigate autogenous shrinkage without a reduction in strength. This study was a continuation of the study by Zhutovsky mentioned earlier. The goal here was to optimize porosity, size, and spacing of lightweight aggregate to achieve sufficient internal curing with a minimal amount of substitution so that the lightweight aggregate could be considered an additive rather than a bulk replacement of conventional aggregate. Two types of fine pumice lightweight aggregate were used, sieved into three uniform sizes. Mixes were designed to have enough lightweight aggregate replacement to provide adequate water for internal curing to prevent autogenous shrinkage for one week after batching. Results showed that porosity of the aggregate, not aggregate spacing, was the governing factor to prevent autogenous shrinkage. Furthermore, additions of relatively small amounts of fine, lightweight aggregate with volume porosity of 50% can eliminate autogenous shrinkage with only minor reductions in compressive strength.

Geiker et al. (2002) looked at mitigating autogenous shrinkage by internal curing using saturated lightweight fine aggregate and the addition of superabsorbent polymer particles. The need for this study is the recent trend of high-performance concrete with low water-to-cement ratios that have tendency to undergo early-age cracking leading to long-term durability issues. In this study, mortars with water-to-cementitious ratios of 0.35 with an 8% replacement by mass of silica fume were evaluated. Mixtures of 8% and 20% replacement of sand with fine lightweight aggregate and 0.04% addition of superabsorbent polymer particles were created. The internal relative humidity was measured in addition to the autogenous deformation under sealed conditions using a custom-built dilatometer immersed in a constant temperature polyalkylene glycol bath. Results showed that each of the three internal curing mixtures either significantly reduced or eliminated autogenous shrinkage. The 20% replacement of lightweight aggregate provided the most extra curing water and resulted in autogenous expansion. A theoretical model is also evaluated to show the effect of aggregate particle spacing on shrinkage throughout the concrete matrix. Both the model and experimental results show that self-desiccation is prevented within approximately 100 μm from an internal curing source, resulting in autogenous deformation of all mixtures except the 20% lightweight aggregate replacement. The water content and spatial distribution of the aggregate throughout the concrete matrix have a significant affect on internal curing and autogenous shrinkage.

CHAPTER 3 - Lightweight Aggregate

Structural lightweight concrete is achieved through use of lightweight aggregate, usually expanded shale, clay, or slate. This type of aggregate has proven to be strong, durable, and economical for several applications. However, lightweight aggregate has some significant differences from normal-weight aggregate and its unique properties must be properly accounted for in concrete mix designs, batching, and placement.

Properties and Background

Types of lightweight aggregates include Vermiculite, Perlite, pumice, scoria, expanded shale, clay, slate, fly ash, and slag. Typically, a correlation exists between density of the aggregate and compressive strength of the concrete. Lower density aggregates are primarily used for insulating or moderate-strength concrete applications. Higher density aggregates, such as expanded shale, clay, and slate, which yield higher strength concrete, are used for structural lightweight concrete applications (ACI Committee 213, 1999). Structural lightweight aggregate concrete is defined by a minimum compressive strength of 2500 psi at 28 days. However, many of these aggregates are capable of producing much higher compressive strengths. Figure 3.1 shows the approximate 28-day, air-dry unit weight range of three types of lightweight aggregate concrete and the use of which each type is generally associated (ACI Committee 213, 1999).

Since structural lightweight concrete is usually composed of expanded shale, clay, or slate, this manuscript will focus on the properties and production process of these materials. It should be noted that the term "shale" is commonly used to encompass lightweight aggregates processed from shale, clay, or slate. Furthermore, the term "all-lightweight" refers to concrete in which both the coarse and fine aggregates are lightweight, whereas the term "sand-lightweight" indicates concrete with coarse lightweight aggregate and normal-weight fine aggregate. All of the concrete mixtures evaluated in this study are sand-lightweight.

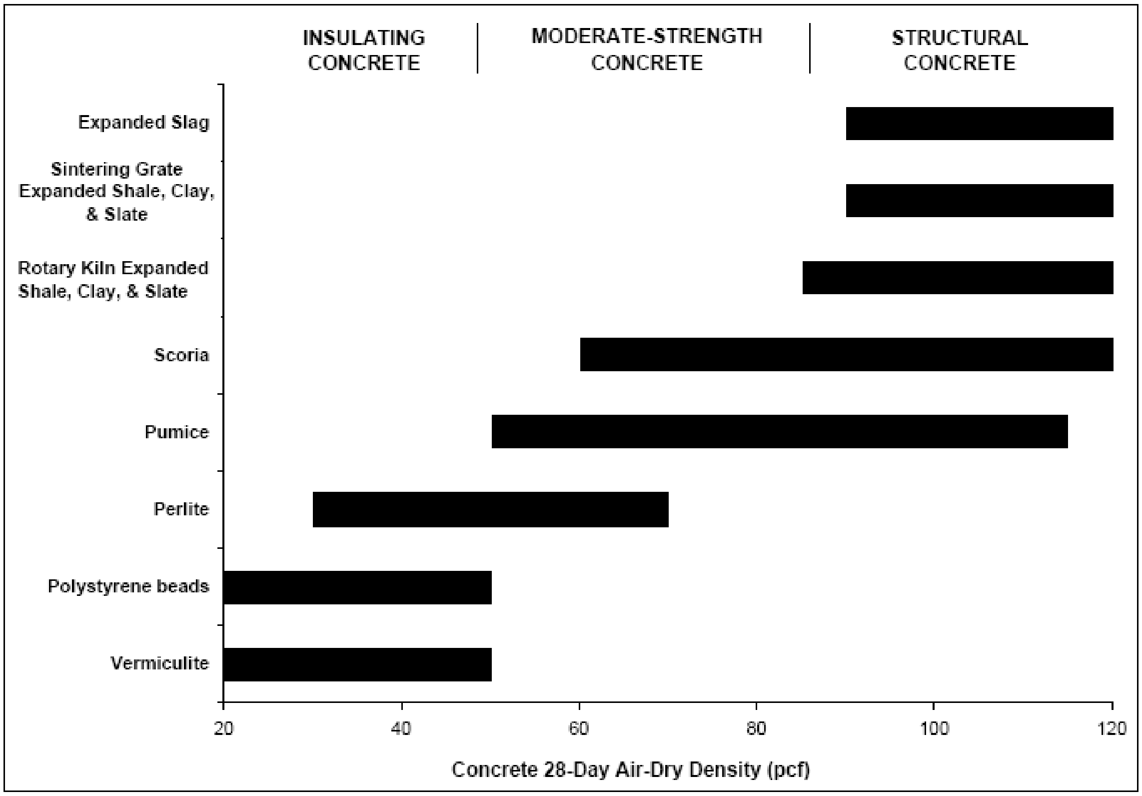

Figure 3.1. Approximate 28-day, air-dry unit weight range of lightweight aggregate.

Production Process

Raw materials used for structural lightweight aggregates are usually from deposits of naturally occurring shale, clay, or slate that are highly siliceous. Two types of production processes are the rotary-kiln method and the sintering method. Both processes heat the raw materials which then expand. The expansion is caused by gas-forming minerals within the materials that liberate gases when the material is heated to the point that it becomes soft and pliable, but not completely melted. These internal gas bubbles create a mass of unconnected air cells which are retained after the material cools. Thus, the raw material is expanded and lower density aggregate is achieved (Expanded Shale, Clay and Slate Institute, 1971).

Rotary-Kiln Method

A rotary kiln is a long cylinder, located on a slight incline that is lined with refractory material and rotates slowly. In this method, shale is crushed and enters at the upper end of the kiln where it is slowly heated and progresses to the lower burner end. The material reaches a temperature of 1800 to 2200°F where it becomes plastic

(Expanded Shale, Clay and Slate Institute, 1971). Internal gases cause the material to expand. Viscosity of the heated material is sufficient to entrap the gases and the internal cellular structure is formed. This internal structure is retained when the material is cooled as a vitrified hard material (ACI Committee 213, 1999).

There are a few different variations to the rotary-kiln process. In one variation, the expanded material is discharged, cooled, then crushed and sized to fit the needed gradation. The next variation pre-sizes and crushes the raw material before it enters the kiln, resulting in expanded sizes that do not need to be re-crushed. The third variation consists of pelletizing the fine raw material as a way of pre-sizing before introducing it to the kiln. Combinations of these three variations are found throughout the industry (Expanded Shale, Clay and Slate Institute, 1971).

Sintering Method

In the sintering process, raw clay or shale is mixed with pulverized fuel and burned under controlled conditions on a moving grate. As in the rotary-kiln method, the material is heated to the point of pyroplasticity so that the gases formed within the material cause it to expand. The gases are then trapped in the viscous material, forming the internal cellular structure of the aggregate. Variations of this process include crushing and sizing the material after it is expanded, and pelletizing the raw material before expansion (ACI Committee 213, 1999).

The rotary-kiln method is more widely used. The Expanded Shale, Clay, and Slate Institute estimates that the rotary-kiln method is used for more than 80% of the structural lightweight concrete that is placed (1971). All three of the aggregates used in this research project were produced using the rotary-kiln method.

Aggregate Properties

Properties of the lightweight aggregate will have an effect on the properties of the lightweight concrete in the plastic and hardened states. Aggregate properties that may affect the concrete include particle shape and surface texture, bulk specific gravity, unit weight, maximum size, strength of aggregate, moisture content and absorption. The effects of many of these properties are similar to those of normal-weight aggregates.

Aggregate properties that are especially peculiar for lightweight aggregates are unit weight and absorption.

Particle Shape and Surface Texture

Lightweight aggregate from different sources or produced by different methods can have a wide variety of particle shapes and surface textures. Factors such as workability, course-to-fine aggregate ratio, cement content, and water content may be influenced. These effects are similar to that of normal-weight aggregate (ACI Committee 213, 1999).

Bulk Specific Gravity and Unit Weight

The specific gravity of lightweight aggregate is significantly lower than normal-weight aggregate due to the internal cellular structure. This decrease in density of lightweight aggregate is how the decrease in unit weight of lightweight concrete is achieved. The bulk specific gravity of lightweight aggregate ranges from 30 to 65 pcf in the dry, loose condition, compared to normal-weight aggregate which ranges from 75 to 110 pcf or more. The specific gravity of lightweight aggregate ranges from 1.1 to 2.2 (Expanded Shale, Clay and Slate Institute, 1971). In addition, the specific gravity of lightweight aggregate varies with particle size; coarse particles are lighter and fine particles are heavier (ACI Committee 213, 1999). Furthermore, the specific gravity of the lightweight aggregate will also vary significantly with the percent of absorption. This variation in specific gravity makes concrete mix design by the absolute volume method more difficult.

Maximum Aggregate Size

The maximum size of lightweight aggregate generally available is ¾ in. or ½ in., depending on the aggregate source. As with normal-weight concrete, maximum aggregate size affects workability, coarse-to-fine aggregate ratio, cement content, optimum air content, drying shrinkage, and potential strength ceiling. With lightweight aggregate, strength of the aggregate is inversely proportional to the aggregate size. Larger aggregates are lower density but also lower strength. The “strength ceiling” of lightweight aggregate refers to the maximum compressive strength achieved in a concrete mixture containing the same aggregate and a given amount of cement. The strength ceiling is reached when the cement content of the mixture is increased, but the compressive strength does not increase. Some lightweight aggregates have high-strength ceilings, similar to that of normal-weight aggregate. However, due to the aggregate size and strength relationship, it has been found that the strength ceiling can be raised for a given lightweight aggregate if the maximum size of the coarse aggregate is reduced (ACI Committee 213, 1999).

Absorption and Moisture Content

Due to the cellular structure of lightweight aggregates, they are usually capable of absorbing significantly more water than normal-weight aggregates. A typical 24-hour absorption test for lightweight aggregates usually results in absorption values of 5 to 20% by weight of dry aggregate (ACI Committee 213, 1999). Absorption values for normal-weight aggregate do not usually exceed 2%. In addition, lightweight aggregate will continue to absorb water for an extended time period of weeks or even months. Rate and amount of absorption depend on the pore structure of the aggregate. Usually, larger aggregate particles will absorb more and at a higher rate for a given aggregate source. An important point to clarify is that the lightweight aggregate absorbs the water into the internal cellular structure of the aggregate particle. The water is not on the outer surface of the aggregate particle, and it is not available to the cement for hydration during mixing and placement. In fact, most of the time even pre-wetted lightweight aggregate will continue to absorb water during mixing, so that the mixture actually contains less water than is intended. Since such high values of absorption are obtained with lightweight aggregates, it is a recommended that the aggregate be pre-soaked or pre-wetted prior to batching. Aggregate stockpiles are usually pre-wetted for at least seven to 14 days. Vacuum-saturated aggregate is also widely available and can significantly improve batch consistency. Whatever method is used, moisture content and absorption need to be accurately accounted for in concrete mixture design and proportioning.

Research Lightweight Aggregate

In this research project, three sources of lightweight aggregate were evaluated. Two of these sources were local Kansas aggregate from the Buildex Company. One of the Buildex sources came from Marquette, Kansas, and the other from New Market, Missouri. The third aggregate source was from the Stalite Company located in North Carolina. Many of the aggregate properties were obtained from the manufacturing companies. However, several of the aggregate properties were verified and compared to KDOT standards.

Research Aggregate Background

Three sources of lightweight aggregate were evaluated. The two local sources produced by the Buildex Company, were both expanded shale. The quarries are located in Marquette, Kansas, and New Market, Missouri. The third aggregate source was expanded slate from North Carolina produced by the Stalite Company. Since there were two aggregate sources from Buildex, these two are referred to as Marquette and New Market.

Buildex - Marquette Aggregate

Marquette, Kansas, is located about 30 miles southwest of Salina, Kansas. The aggregate from here is expanded shale and was obtained in the ASTM blend size of ½" x No. 4. This gradation was selected due to availability of aggregate. A picture of the Marquette aggregate is shown in Figure 3.2. Buildex manufacturing specifications report that the Marquette ½" x No. 4 aggregate has a bulk specific gravity of 1.09, a loose density of 37 pcf at 6% moisture content, and a saturated density of 52 pcf when the aggregate stockpile is pre-wetted for seven to 14 days (Buildex Inc., 2006). Particle shape and surface texture of the Marquette aggregate is angular and rough.

Figure 3.2. Buildex-Marquette lightweight aggregate.

Pictures were also taken using a scanning-electron microscope to evaluate the pore structure of the lightweight aggregate. Pictures of the Marquette aggregate are shown in Figure 3.3 and Figure 3.4 with 400μm and 50μm scales, respectively. At the 400μm scale, the surface roughness and texture is shown. This property allows for better bond between the aggregate and the paste matrix within the contact zone. Furthermore, the intricate pore structure of the lightweight aggregate can be seen at the 50μm scale. This pore structure is responsible for the high absorption capacity of the lightweight aggregate. Voids of the Marquette aggregate pore structure appear larger and more frequent when compared to the New Market and Stalite aggregates. This property likely has a correlation to the Marquette aggregate having the highest initial absorption rate, discussed later, of all three lightweight aggregates.

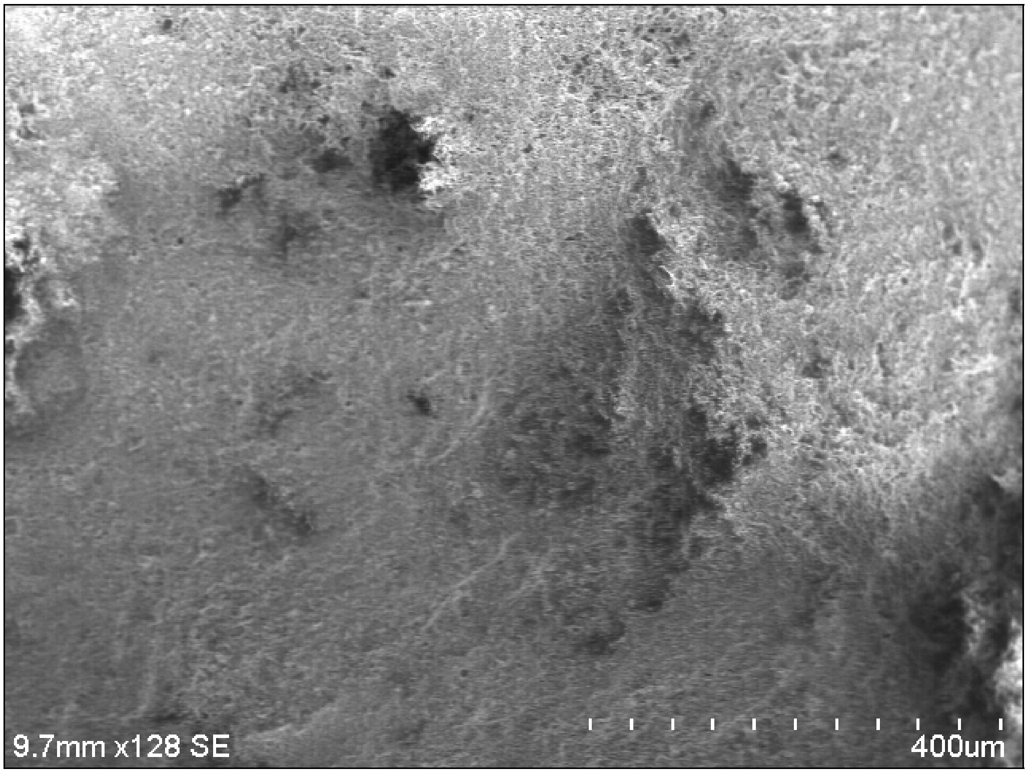

Figure 3.3. Marquette aggregate with 400μm scale using the scanning-electron microscope.

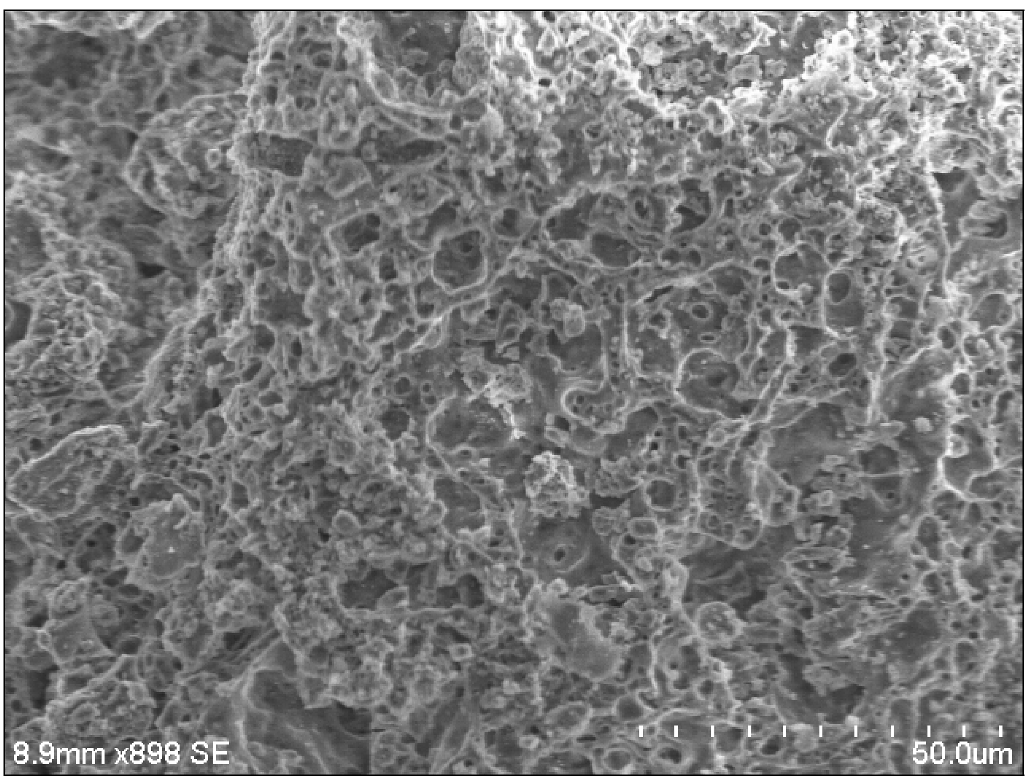

Figure 3.4. Pore structure of Marquette aggregate with 50μm scale using scanning-electron microscope.

Buildex-New Market Aggregate

New Market, Missouri, is located about 30 miles northwest of Kansas City. The aggregate from here is expanded shale and was obtained in the ASTM blend size of ½" x No. 4. A picture of the New Market aggregate is shown in Figure 3.5. Unlike the Marquette aggregate, the New Market rock is available pre-saturated by vacuum saturation. Buildex manufacturing specifications report that the New Market rock has a bulk specific gravity of 1.15, a loose density of 43 pcf at a 6% moisture content, and a vacuum-saturated density of 54 pcf (Buildex Inc., 2006). Overall, the New Market aggregate has smaller, more rounded aggregate particles and the surface texture appears to be smoother.

Figure 3.5. Buildex-New Market lightweight aggregate.

Pictures of the New Market aggregate using the scanning-electron microscope are shown in Figure 3.6 and Figure 3.7 with 400μm and 50μm scales, respectively. At the 400μm scale, the surface roughness and texture is shown. As shown, the New Market aggregate is the smoothest of all three lightweight aggregates evaluated. However, the uneven surface texture still allows for better bond within the contact zone of the aggregate and paste matrix. In addition, the voids of the lightweight aggregate pore structure are shown with the 50μm scale. The voids within the pore structure of the New Market aggregate do not seem to occur as frequently as with the Marquette aggregate. However, since the New Market aggregate has the highest absorption capacity, but a lower absorption rate, as discussed later, it can be deduced that the internal pore structure of the New Market aggregate has a greater volume of voids with smaller or less frequent openings.

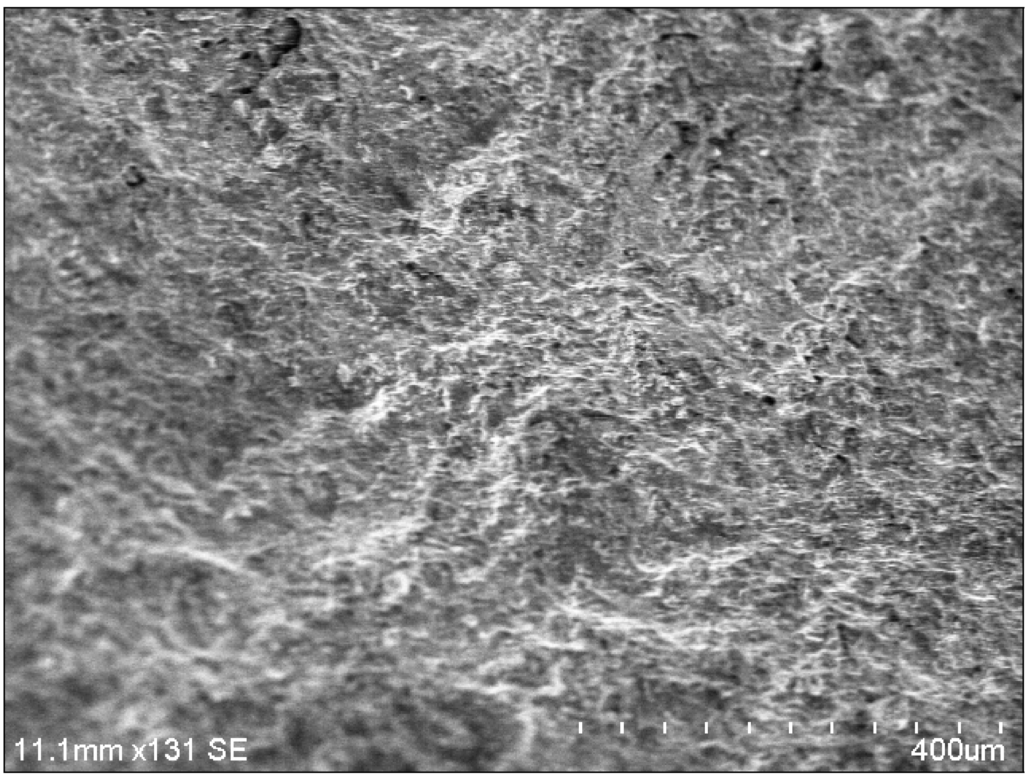

Figure 3.6. New Market aggregate with 400μm scale using the scanning-electron microscope.

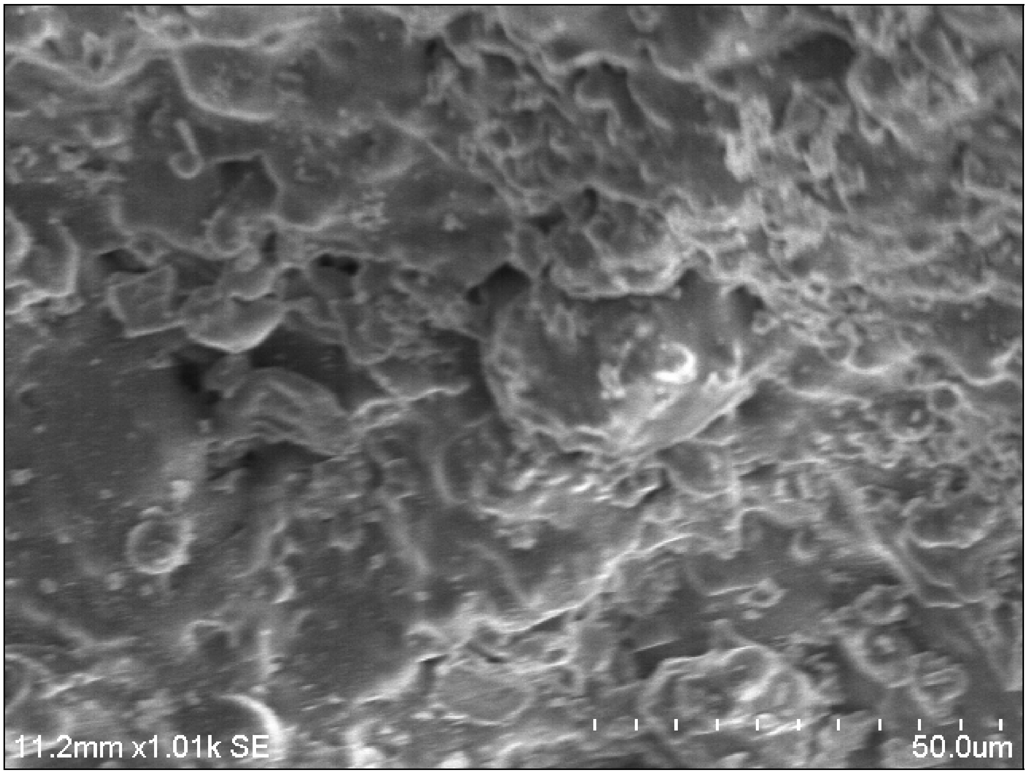

Figure 3.7. Pore structure of New Market aggregate with 50μm scale using scanning-electron microscope.

Stalite Aggregate

The Carolina Stalite Company is located 40 miles east of Charlotte in the foothills region of North Carolina. This aggregate is expanded slate. Even though shipping costs would likely cause the Stalite aggregate to not be economical for KDOT, it was chosen for its high-quality reputation to serve as a control case or comparison standard to evaluate the other lightweight aggregates. In addition, if performance of the Stalite aggregate proved to be far better than the local Kansas aggregate, cost and benefit of shipping the aggregate to Kansas on a large scale would be assessed. Stalite manufacturing specifications report that the bulk specific gravity of their rock is 1.45 in the dry condition and 1.52 in the saturated surface dry (SSD) condition. Loose density in the dry condition is 50 pcf and 52 pcf in the SSD condition (Harmon, 2007). The Stalite aggregate was the coarsest grade of the three tested; particle shape was angular; and surface texture was smoother than the Marquette aggregate, but rougher than the New Market aggregate. A picture of the Stalite aggregate is shown in Figure 3.8.

Figure 3.8. Stalite lightweight aggregate.

Pictures of the Stalite aggregate, using the scanning-electron microscope are shown in Figure 3.9 and Figure 3.10 with 400μm and 50μm scales, respectively. At the 400μm scale, the surface roughness and texture of the Stalite aggregate is shown. The Stalite aggregate appears to be between the surface roughness and texture of the Marquette and New Market aggregates. Furthermore, the voids within the pore structure of the Stalite aggregate appear to be fewer and more spaced. This pore structure likely correlates to the Stalite aggregate having a much lower absorption capacity and absorption rate compared to the Marquette and New Market aggregates, as discussed later.

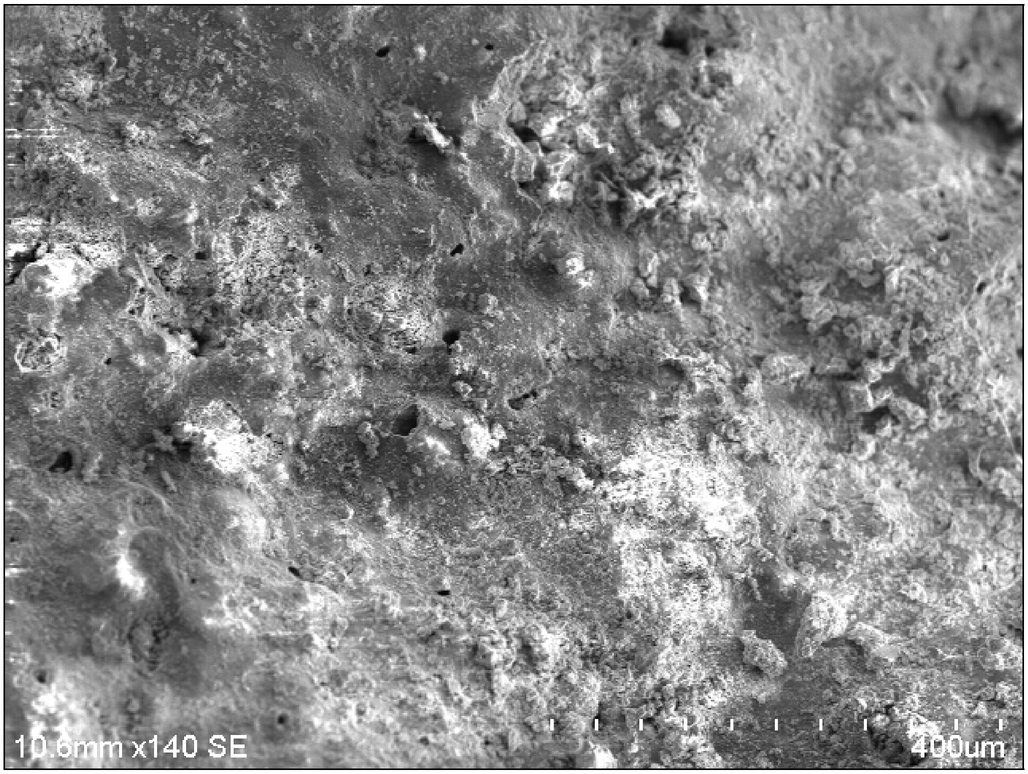

Figure 3.9. Stalite aggregate with 400μm scale using the scanning-electron microscope.

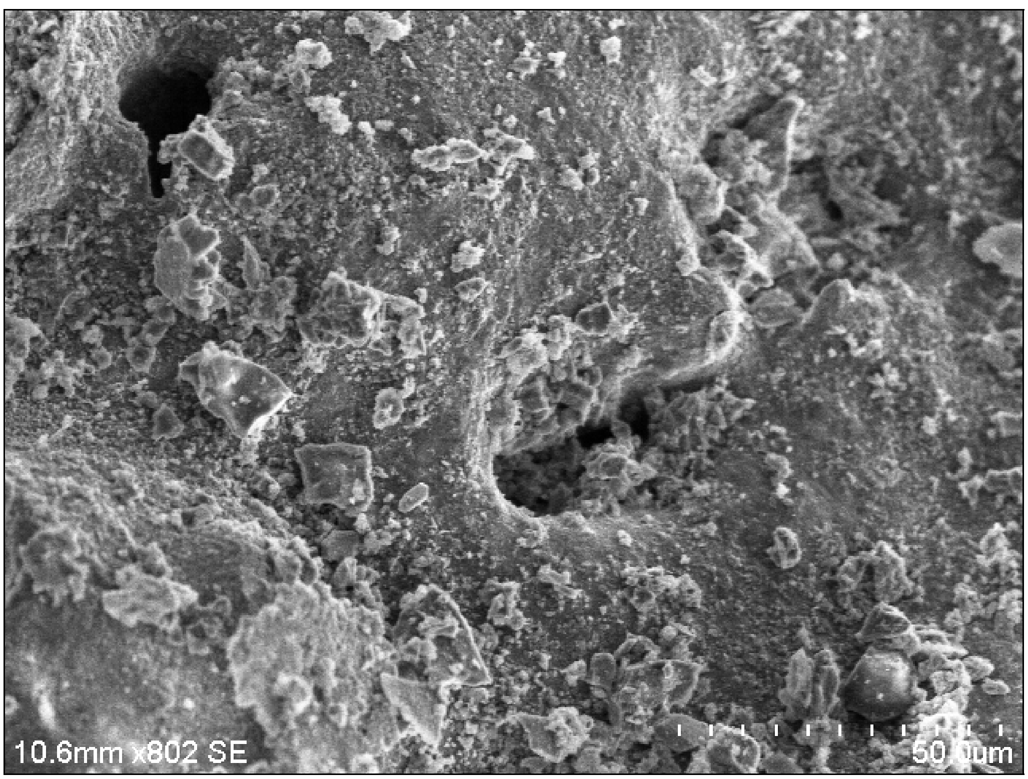

Figure 3.10. Pore structure of Stalite aggregate with 50μm scale using scanning-electron microscope.

Course Aggregate Gradation

To verify the gradations received from Buildex and Stalite, a sieve analysis was conducted according to ASTM C136. This gradation was compared to current KDOT specifications for a CA-4 coarse aggregate gradation. The sieve analysis for the Marquette aggregate is shown in Table 3.1. The corresponding gradation curve is shown, along with upper and lower limits for the KDOT CA-4 specification, in Figure 3.11. As shown, gradation of the Marquette aggregate was well within KDOT CA-4 limits.

Table 3.1. Sieve analysis for Marquette aggregate.

Sieve	Material Weight (g)	Percent Retained	Cumulative Percent Retained	Percent Passing	KDOT Low Percent Passing	KDOT High Percent Passing
3/4”	33	1.1	1.1	99.0	100.0	100.0
1/2”	396	12.6	13.6	86.4	65.0	100.0
3/8”	849	27.0	40.6	59.4	30.0	70.0
# 4	1736	55.1	95.7	4.3	0.0	25.0
# 8	86	2.7	98.4	1.6	0.0	5.0
# 16	15	0.5	98.9	1.1	0.0	0.0
# 30	6	0.2	99.1	0.9	0.0	0.0
# 50	7	0.2	99.3	0.7	0.0	0.0
# 100	5	0.2	99.5	0.5	0.0	0.0
pan	17	0.5	100.0	0.0	0.0	0.0

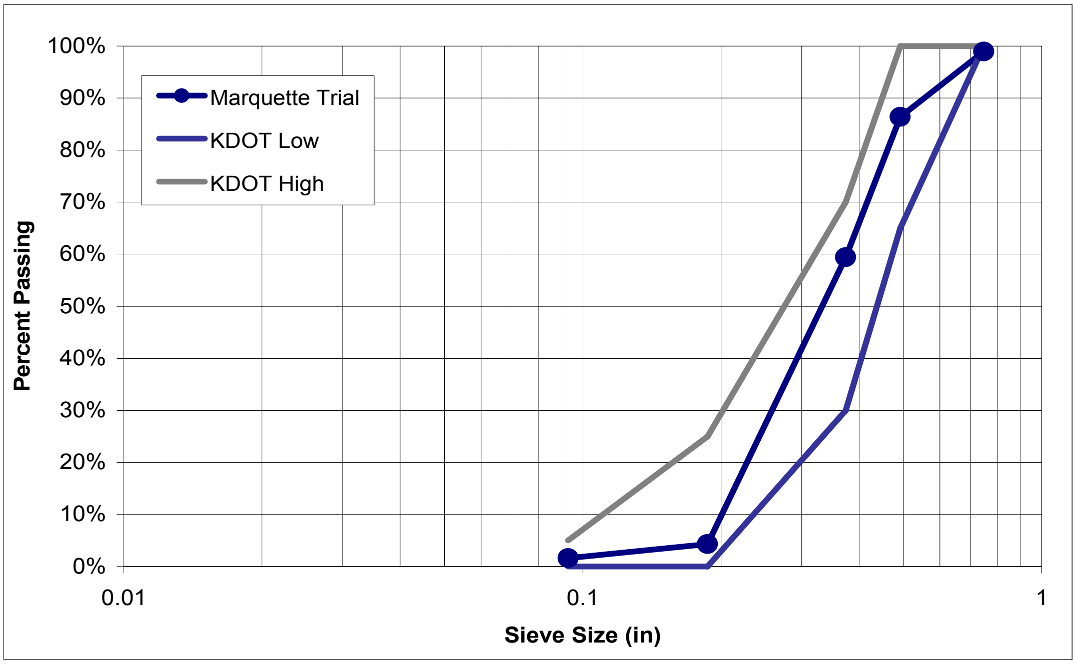

Figure 3.11. Gradation of Marquette aggregate with KDOT CA-4 range.

The sieve analysis for the New Market aggregate is shown in Table 3.2. Gradation of the New Market rock and KDOT CA-4 specification limits are shown in Figure 3.12. This aggregate did not completely fall within CA-4 limitations but was not too far out of range, and the mixed aggregate gradation, shown in Chapter 4, fell within KDOT limits.

Table 3.2. Sieve analysis for New Market aggregate.

Sieve	Material Weight (g)	Percent Retained	Cumulative Percent Retained	Percent Passing	KDOT Low Percent Passing	KDOT High Percent Passing
3/4”	0	0.0	0.0	100.0	100.0	100.0
1/2”	137	4.2	4.2	95.8	65.0	100.0
3/8”	520	15.8	20.0	80.0	30.0	70.0
# 4	182	55.6	75.6	24.4	0.0	25.0
# 8	645	19.6	95.2	4.8	0.0	5.0
# 16	80	2.4	97.7	2.3	0.0	0.0
# 30	23	0.7	98.4	1.6	0.0	0.0
# 50	14	0.4	98.8	1.2	0.0	0.0
# 100	6	0.2	99.0	1.0	0.0	0.0
pan	34	1.0	100.0	0.0	0.0	0.0

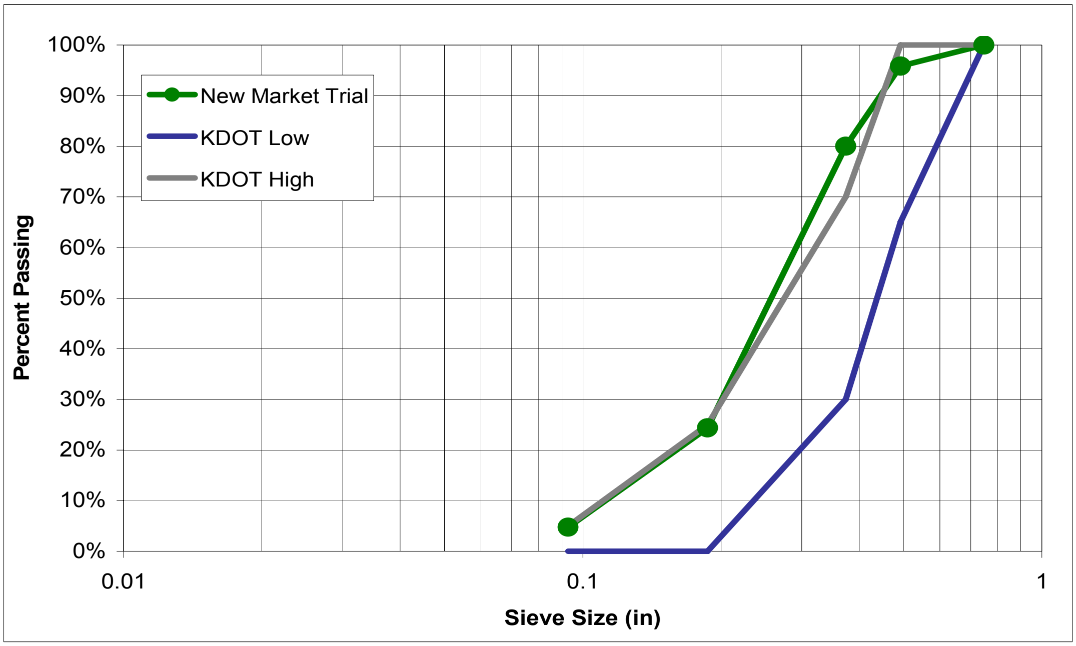

Figure 3.12. Gradation of New Market aggregate with KDOT CA-4 range.

The sieve analysis of the Stalite aggregate is given in Table 3.3. A gradation curve of this aggregate and KDOT CA-4 specification limits are shown in Figure 3.13. It can be seen that the Stalite rock was more coarsely graded than the other two aggregates being studied. For this reason, the Stalite aggregate fell outside CA-4 limits. However, like the New Market aggregate, the mixed gradation of the Stalite aggregate, shown in Chapter 4, fell within KDOT specification limits.

Table 3.3. Sieve analysis for Stalite aggregate.

Sieve	Material Weight (g)	Percent Retained	Cumulative Percent Retained	Percent Passing	KDOT Low Percent Passing	KDOT High Percent Passing
3/4”	227	6.1	6.1	93.9	100.0	100.0
1/2”	2191	58.6	64.6	35.4	65.0	100.0
3/8”	835	22.3	86.9	13.1	30.0	70.0
# 4	362	9.7	96.6	3.4	0.0	25.0
# 8	37	1.0	97.6	2.4	0.0	5.0
# 16	13	0.4	97.9	2.1	0.0	0.0
# 30	8	0.2	98.2	1.8	0.0	0.0
# 50	13	0.4	98.5	1.5	0.0	0.0
# 100	13	0.4	98.9	1.2	0.0	0.0
pan	43	1.2	100.0	0.0	0.0	0.0

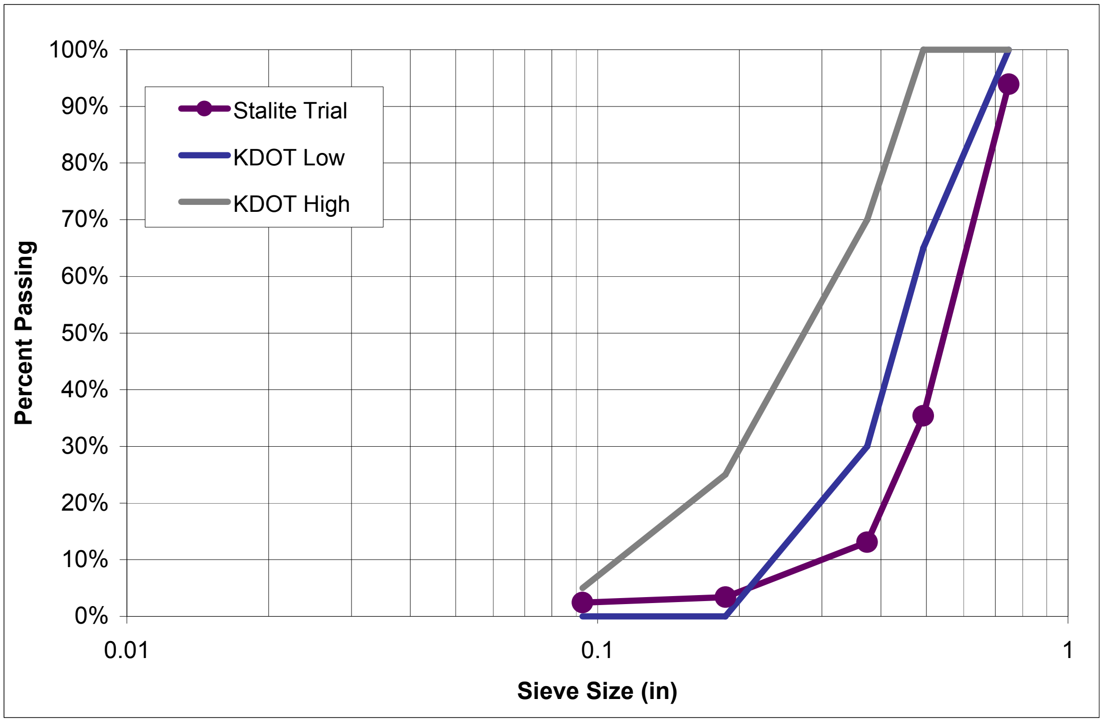

Figure 3.13. Gradation of Stalite aggregate with KDOT CA-4 range.

Absorption

Like many types of lightweight aggregate, the three aggregates in this study absorbed more water than normal-weight aggregate. The Stalite aggregate had a maximum absorption of approximately 6%. However, the Marquette and New Market aggregates had much higher absorption values. The Marquette rock had a maximum absorption around 25%, whereas vacuum-saturated New Market rock had an absorption value of 28%. Absorption values of this magnitude can cause great difficulty with proportioning of the mix design and batching if not properly accounted for. Furthermore, absorption rates of the Marquette and New Market aggregates were also considerably high compared to normal-weight aggregate with a maximum absorption around 2%.

To determine the percent absorption and absorption rate of the aggregate, a test was conducted similar to the absorption test of AASHTO T85. The aggregate was oven dried, then submerged in water in a hanging basket hooked to a scale, with weight measured periodically. The main modification to the AASHTO T85 absorption test is that this test was not stopped at a time period of 24 hours, but instead continued for 90 days. Weight measurements were recorded several times during the first 24 hours, then at least daily for two weeks, and then periodically up to 90 days. The percentage absorption was then determined. All absorption values were evaluated from the oven-dry state since only the New Market aggregate was available vacuum saturated. It was believed that the vacuum-saturated aggregate would not represent the worst case or most extreme scenario. Therefore, no vacuum-saturated aggregate was used during the research project. The percent absorption versus time for the first 24 hours after submersion is shown in Figure 3.14.

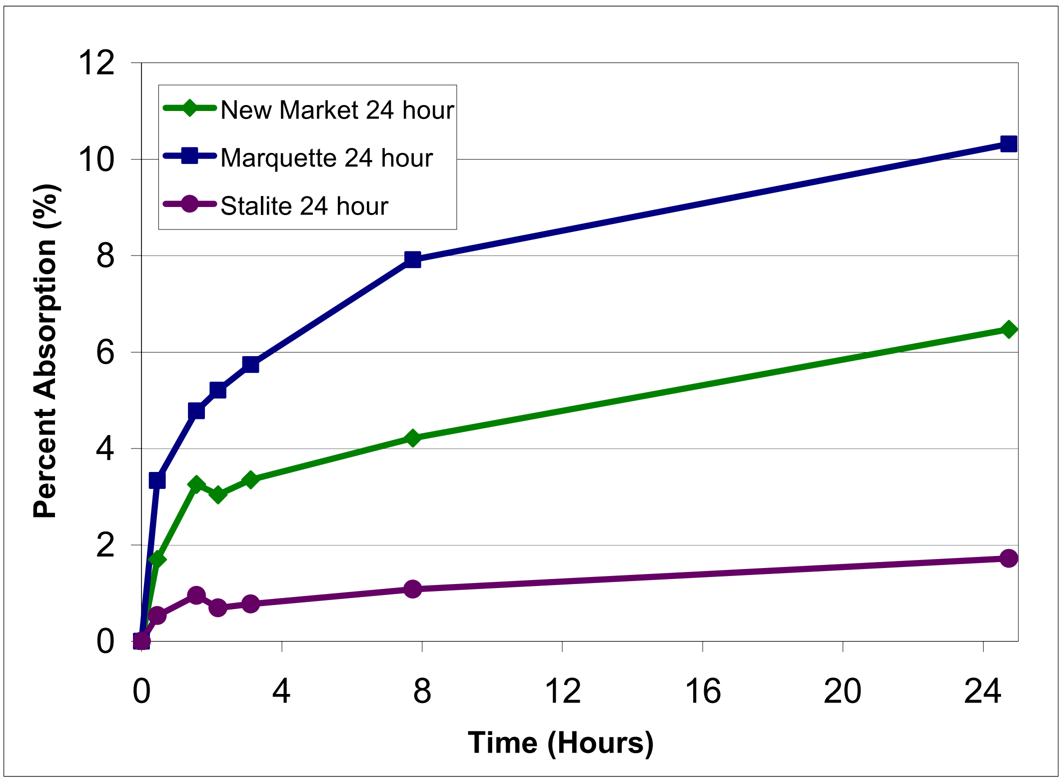

Figure 3.14. Percent absorption for the first 24 hours after submersion.

As shown in the graph, the 24-hour absorption values for the New Market, Marquette, and Stalite aggregates were 6.5%, 10.3%, and 1.7%, respectively. Another notable fact was the rate of increase of all three aggregates. In the first 24-hour period, while all three appeared to be in a parabolic trend and beginning to level off, they also all seemed to still be increasing at a significant rate. A graph of the percent absorption versus time for the entire 90-day submersion period is shown in Figure 3.15. Maximum absorption values obtained were 30.1% for New Market, 25.9% for Marquette, and 8.1% for Stalite. Actual percentages for 1, 3, 7, 14, 28, 60, and 90 days are given in Table 3.4. As shown, absorption values for the New Market and Marquette aggregates were significantly higher than for the Stalite. Another important difference was absorption rates. The Stalite absorption leveled off in approximately the same time period as the Marquette, but at a much lower magnitude. The Marquette absorption increased at a faster rate than did the New Market. However, the New Market rock absorption rate did not level off and instead continued to increase steadily throughout the 90-day period. The New Market absorption did appear to be leveling off towards the end of the 90-day test.

These high absorption values need to be accounted for in the mix design proportioning, and the absorption rates need to be accounted for during the batching process. If the mix design is not adjusted properly for moisture content and absorption of the aggregate going into the batch, the workability, water-to-cement ratio, yield, and strength will greatly be affected. In addition, if any of these three lightweight aggregates is batched in the dry condition, they will absorb a significant amount of mixing water which will also affect the workability, water-to-cement ratio, and strength. Details of how absorption and moisture content values were determined and used for this research project are given in Chapter 4.

Table 3.4. Percentage absorption by weight.

	Percent Absorption by Weight		
Day	**New Market**	**Marquette**	**Stalite**
1	6.5	10.3	1.7
3	10.2	14.4	3.1
7	13.4	17.5	4.0
14	16.6	20.6	4.9
28	21.1	23.1	6.0
60	26.1	24.8	7.0
90	28.8	25.5	7.7

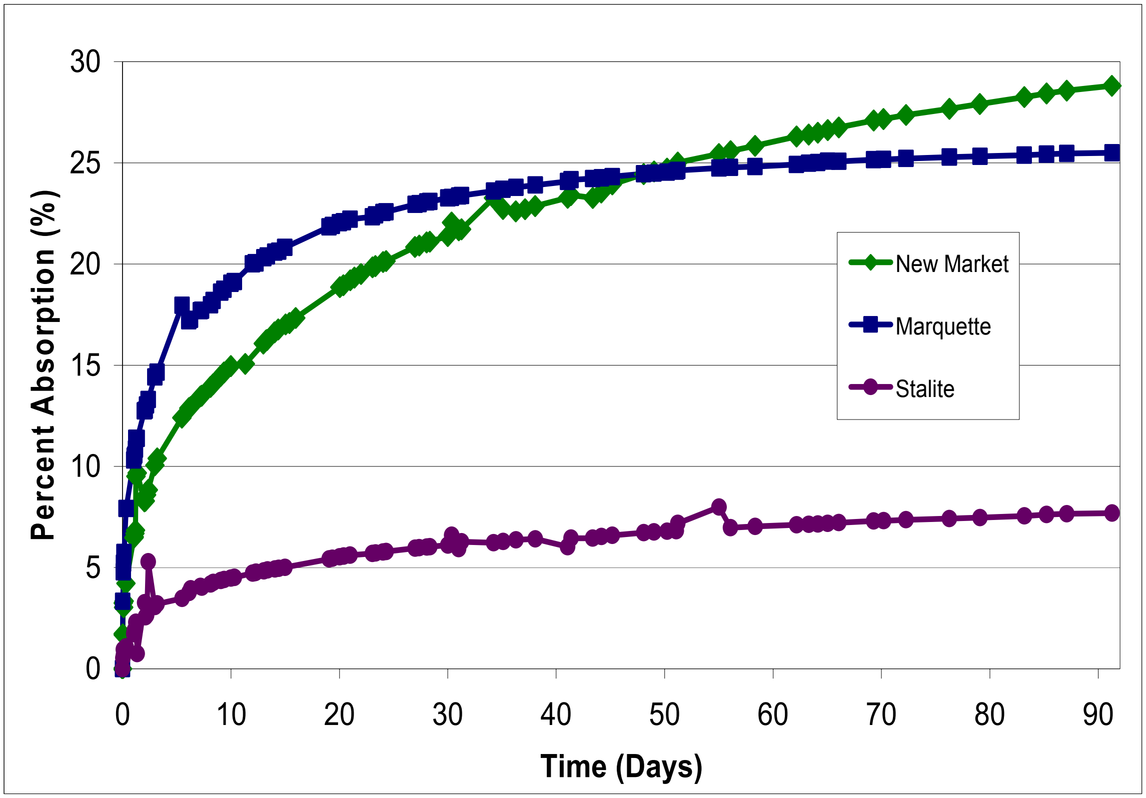

Figure 3.15. Percent absorption for the 90-day submersion period.

L.A. Abrasion

Resistance to degradation of coarse aggregate by abrasion and impact can be measured using the Los Angeles machine and the AASHTO T96 method or the KDOT KTMR25 specification. In this test, a standard aggregate grading is subjected to a combination of abrasion or attrition, impact, and grinding in a rotating steel drum containing a specified number of steel spheres. The test continues until the drum has reached 500 revolutions. The aggregate sample is then removed, sieved, and washed. The remaining aggregate, larger than the No. 12 sieve, is then oven dried and weighed. The degradation is then measured by the percent loss. All three lightweight aggregates in this study were subjected to the L.A. Abrasion test. Gradation designation used for each aggregate was grading C, using 2500g of aggregate retained on both the 1/4" and #4 sieves. Results are shown in Table 3.5. According to these results, the Stalite aggregate displayed the best resistance to degradation by abrasion and impact with a percent loss of

24%. However, the Marquette and New Market aggregates were not significantly worse with percent loss values of 27% and 31%, respectively. The KDOT limit for KTMR25 in section 1102 for percent loss of degradation due to freezing and thawing is 50%. All three lightweight aggregates are within this limit.

Table 3.5. Resistance to degradation of coarse aggregate in the Los Angles machine.

Aggregate Type	Percent Loss
Marquette	27
New Market	31
Stalite	24

CHAPTER 4 - Lightweight Concrete Mixture Proportioning and Design

When determining proportions of a concrete mixture, several factors need to be considered such as desired concrete properties, materials, and placement methods. The material of interest in this case is, of course, the three types of lightweight aggregate being researched. Special consideration was given to pre-wetting the aggregate and subsequently accounting for the moisture content and absorption values in the mix designs. Two different optimized mix designs, one for a bridge deck and one for prestressed beams, were created for each type of lightweight aggregate. Each of these optimized mixtures was then used to conduct all material property tests to compare against KDOT standards.

Absolute Volume Method

The absolute volume method of proportioning concrete was used to create the concrete mix designs for this project. The principle behind this method is to design one cubic yard of concrete based on the volume of the constituents. This consists of setting initial values of weight for cement, water, and coarse aggregate, then using the specific gravity of these materials to convert these weights into volumes. The volume of air assumed to be in the mixture is also calculated. These volumes are summed and the rest of the volume within the one cubic yard being designed for is filled up with fine aggregate. Weight of the fine aggregate can then be determined using specific gravity and the calculated volume.

This is a common proportioning method and has proven to be successful and accurate with normal-weight concrete. However, problems arise when using this method with lightweight aggregate. The issue in this case stems from the nature of the specific gravity of the lightweight aggregate. Specific gravity of lightweight aggregate varies with particle size, where coarse particles are lighter and fine particles are heavier. In addition, specific gravity of lightweight aggregate changes as absorption increases. Furthermore, specific gravity of lightweight concrete cannot be accurately determined using AASHTO T85, which states it is not intended to be used with lightweight aggregates. For these reasons, specific gravity values used in the mix designs for this project were obtained from Buildex and Stalite. Each of these companies had determined an average specific gravity value that had been successfully used in numerous concrete mixture designs. These specific gravity values were based on aggregate that had been pre-wetted in a stockpile for seven to 14 days. Although the actual lightweight aggregate specific gravity value likely fluctuated for each concrete batch, using average values from Buildex and Stalite proved to produce consistent results, and was therefore used throughout the project. Specific gravity values used were 1.52 for New Market, 1.44 for Marquette, and 1.52 for Stalite. For this project, an Excel worksheet was created according to the absolute volume method to facilitate mix design calculations. A sample concrete mix design worksheet is shown in Figure 4.1.

An alternative concrete mixture proportioning method is recommended by Buildex. This method consists of initially batching the materials required to achieve a certain strength based on the loose bulk density of the pre-wetted aggregate. With this method, material proportions are largely based on empirical data and experience. Air content and yield of the mixture is then determined, and the proportion of the coarse lightweight aggregate is then adjusted to achieve the desired yield and unit weight. This method has been used by several batch plants and has been shown to successfully produce consistent concrete mixes. However, KDOT requires that the water-to-cement ratio be less than or equal to specified maximum values, 0.44 for a bridge deck and 0.35 for prestressed beams, and this method does not accurately calculate the water-to-cement ratio. For this reason, the absolute volume method was used for concrete mixture proportioning.

Lightweight Concrete

Mix: KDOT Stalite

Date:

w/cm ratio = 0.380

Batch Size = 2.5 ft^3

Material	% Moisture Content	% Absorption	Specific Gravity	Unit Weight (lb/ft^3)	Design lbs/cubic yard	Batch lbs/cubic yard	Volume (ft^3)	% by Volume	Batch Weight			
Water			1	62.4	275.5	292	4.415	-	27.07	lb		
Cement			3.15	196.6	725	725	3.688	-	67.13	lb		
Fly Ash			2.7	168.5	0	0	0.000	-	0.00	lb		
Buildex - Marquette			1.44	89.9		0	0.000	0.0	0.00	lb		
Buildex - KC			1.52	94.8		0	0.000	0.0	0.00	lb		
Staylite	0	0	1.52	94.8	650.5	651	6.858	30.4	60.23	lb	% agg.	0.400
Sand	0	1	2.63	164.1	1688	1688	10.286	54.4	156.30	lb	% agg.	0.600
Air Content (2 fl.oz/100 lb cement)	6.5	%					1.755		0.408282	fl. oz.	11.7	mL

Total Weight = 3355.9 pcy

Total Volume = 27.003 ft^3

Unit Weight = 124.3 pcf

Unit Weight w/out air = 132.9 pcf

slump = ________ in

Measured Weight= 14.6

Unit Weight of actual batch = 122.7 pcf

Actual % air = 7.8 %

Rollameter measured % air = 6.3 %

Figure 4.1. Sample mix design worksheet

Trial Mix Designs

One of the main objectives of this research project was to design and create lightweight concrete mixtures for both bridge deck applications and prestressed beams. The bridge deck mixture was studied first and several mix designs were created and tested until a satisfactory, optimized mix design was achieved. This "optimized" mix design refers to a concrete mixture that had the lowest cement content, best-fit gradation, lowest obtainable unit weight, and still achieved desired workability and compressive strength required by KDOT. After the optimized bridge deck mixture was achieved, a similar process was used to create and test the prestressed beam mixture design.

Concrete Tests

To evaluate the preliminary lightweight concrete mixtures, slump, unit weight, volumetric air content, gravimetric air content, and compressive strength of each mixture was determined. Several concrete variables were altered during the preliminary mix design phase. These fresh and hardened concrete tests permitted the optimum mix design variables to be selected.

Slump

The slump test was conducted on each concrete mixture according to AASHTO T119. During the slump test, fresh concrete was placed in the slump cone in three layers of equal volume and each layer was rodded 25 times. Next, the slump cone was lifted and the plastic concrete sank down. The vertical distance between the original height of the cone and the displaced original center of the concrete was then measured as the slump. This test is a measure of workability and mixture consistency between batches. A picture of the slump test being conducted is shown in Figure 4.2.

Figure 4.2. Conducting the slump test.

Unit Weight

The unit weight, or density, of the freshly mixed concrete was determined according to AASHTO T121. This test consisted of consolidating a sufficient volume of concrete in a rigid container to obtain a specific volume. The weight of the fresh concrete was then measured and the density was the weight divided by the volume. The unit weight was especially important in lightweight concrete since a maximum unit weight is usually specified. For this research project, the unit weight was measured from the bowl of the volumetric air meter. A picture of this test being conducted is shown in Figure 4.3.

Figure 4.3. Unit weight measurement and scale.

Volumetric Air Content

To measure the percent air content in the fresh concrete, the volumetric air meter, also known as a rollometer, was used according to AASHTO T196. It was necessary to use the volumetric method instead of the pressure method (AASHTO T152) because the pressure method cannot accurately determine the air content of lightweight concrete. The pressure method, since it consists of exposing the fresh concrete to pressurized air, causes water and paste to be forced into the pores of the lightweight aggregate, resulting in a much higher and erroneous air content reading. The volumetric method works on the simple principle of volume. A prescribed volume of concrete was used to fill the bowl of the volumetric air meter. The top section of the air meter is then attached and filled with water. The apparatus was then agitated by inverting, shaking, and rolling to break up the mixed concrete. Once the concrete constituents were separated, the air, both entrapped and entrained, was released and rose to the top of the calibrated top section. The percent air content could then be read. A picture of the volumetric air meter is shown in Figure 4.4 and the rollometer being agitated is shown in Figure 4.5 and Figure 4.6. It should be noted that it is considerably more difficult to release the lightweight aggregate during this test than it is for normal-weight aggregate due to the lighter weight of the aggregate. If the fresh concrete is not properly shaken and the constituents freed, then not all of the air is released and the measurement is inaccurate. Extra care should be taken to agitate the apparatus to sufficiently break-up the fresh concrete. In addition, it is suggested that for large projects, gravimetric air content be used to determine air content once the volumetric method has been used to establish a specific gravity for the lightweight aggregate. The volumetric method can be time-consuming and delay placement; therefore, the gravimetric air content has shown to be reliable and more time-efficient.

Figure 4.4. Rollometer used to determine volumetric air content.

Figure 4.5. Agitating rollometer by inverting and shaking.

Figure 4.6. Agitating rollometer by rolling.

Gravimetric Air Content

The gravimetric air content was determined for each concrete mixture according to AASHTO T121. Gravimetric air content is based on calculations of the unit weight of the freshly mixed concrete and specific gravities of the materials used in the mix design. The basic principle is that a certain volume of concrete, composed of a certain ratio of materials, results in a composite density of the material constituents plus air. Composite density is calculated assuming zero air content, and the difference between the actual measured density and the zero air density is the volume of air in the fresh concrete. The percentage air content can then be determined. Theoretically, the gravimetric air content and the volumetric air content should be the same. However, since gravimetric air content calculations are based on the specific gravity of the lightweight aggregate, and the specific gravity of the lightweight aggregate varies due to particle size and absorption, then the air content measured by the volumetric method and the calculated gravimetric air content can be off. When this discrepancy occurs, the volumetric air content should be taken as the real air content. For large projects, it is recommended that volumetric air content first be measured and then specific gravity of the lightweight aggregate be calculated, based on the measured volumetric air content. The gravimetric air content can then be used instead of the volumetric method to avoid concrete placement delays.

Compressive Strength

Compressive strength was determined for all preliminary concrete mixtures according to AASHTO T22. Three 4” x 8” cylinders were used to determine compressive strength at seven and 28 days for the bridge deck mixes and at 16 hours for the prestressed mixes. Specimens were made according to AASHTO T126 procedures. A picture of cylinders being made is shown in Figure 4.7. Some of the preliminary bridge deck mixes were tested using sulfur caps, but most of the bridge deck mixes, and all of the prestressed mixes, were tested using neoprene pads and end caps. A picture of a 4” x 8” cylinder being tested for compressive strength is shown in Figure 4.8. The neoprene caps are shown in the figure, along with a loosely fitted canvas wrap used for safety and to reduce mess.

Figure 4.7. Making compressive cylinders.


Figure 4.8. Testing a compressive cylinder with neoprene caps.

Lightweight Bridge Deck Concrete Mixtures

Several preliminary lightweight concrete bridge deck mixtures were created and tested for slump, unit weight, air content, and compressive strength. The goal was to obtain an optimized bridge deck mixture for each of the three lightweight aggregates being researched that had the lowest cement content, best-fit gradation, lowest unit weight, and that still achieved desired workability and compressive strength.

One mixture goal was a compressive strength of 4000 psi, which correlates to a 5200 psi design, laboratory compressive strength that needed to be achieved (ACI 318, 2004). KDOT requires a minimum of 639 pcy of cement for grade 4.0 concrete for structures, and a maximum water-to-cement ratio of 0.44 (KDOT Section 402). KDOT also has a gradation requirement that needed to be considered. A reasonable workability was desired, correlating to a slump range of 2 to 6 inches. Finally, the unit weight also needed to be considered, but a maximum unit weight was not required to be less than 120 pcf, so that other mix design requirements, such as gradation effects, could be evaluated. KDOT specifications also require an air content of 6.5±1.5% (KDOT Section 402).

Materials

Materials used in the lightweight concrete bridge deck mixes were Type I or Type I/II cement; normal-weight fine aggregate; air-entraining admixture; water; and the three types of lightweight aggregate, Marquette, New Market, and Stalite, discussed in Chapter 3.

Type I cement was obtained from Monarch Cement Company. Two different cement shipments were used throughout the project. The air-entraining admixture was Daravair 1000 from W.R. Grace Admixtures. Dosage rates for the air-entraining admixture were altered until successful percent air contents were achieved. Normal-weight sand was obtained from Midwest Concrete Materials. This sand meets KDOT specifications for fine aggregate. The fine-aggregate sieve analysis is given in Table 4.1 compared to KDOT fine aggregate FA-A limits, and the corresponding gradation curve is shown in Figure 4.9. Since the coarse-to-fine aggregate ratio is varied, the mixed-aggregate gradations are given below in the Coarse-to-Fine Aggregate Ratio section.

Table 4.1. Sieve analysis for normal-weight fine aggregate.

Sieve	Material Weight (g)	Percent Retained	Cumulative Percent Retained	Percent Passing	KDOT FA-A Low Percent Passing	KDOT FA-A High Percent Passing
3/4”	0.0	0.0	0.0	100.0	100.0	100.0
1/2”	0.0	0.0	0.0	100.0	100.0	100.0
3/8”	0.0	0.0	0.0	100.0	100.0	100.0
# 4	150.8	4.4	4.4	95.6	90.0	100.0
# 8	553.2	16.1	20.5	79.5	73.0	100.0
# 16	820.0	23.9	44.4	55.6	45.0	85.0
# 30	810.6	23.6	68.0	32.0	23.0	60.0
# 50	782.9	22.8	90.9	9.2	7.0	30.0
# 100	245.1	7.1	98.0	2.0	0.0	10.0
pan	69.0	2.0	100.0	0.0	0.0	0.0

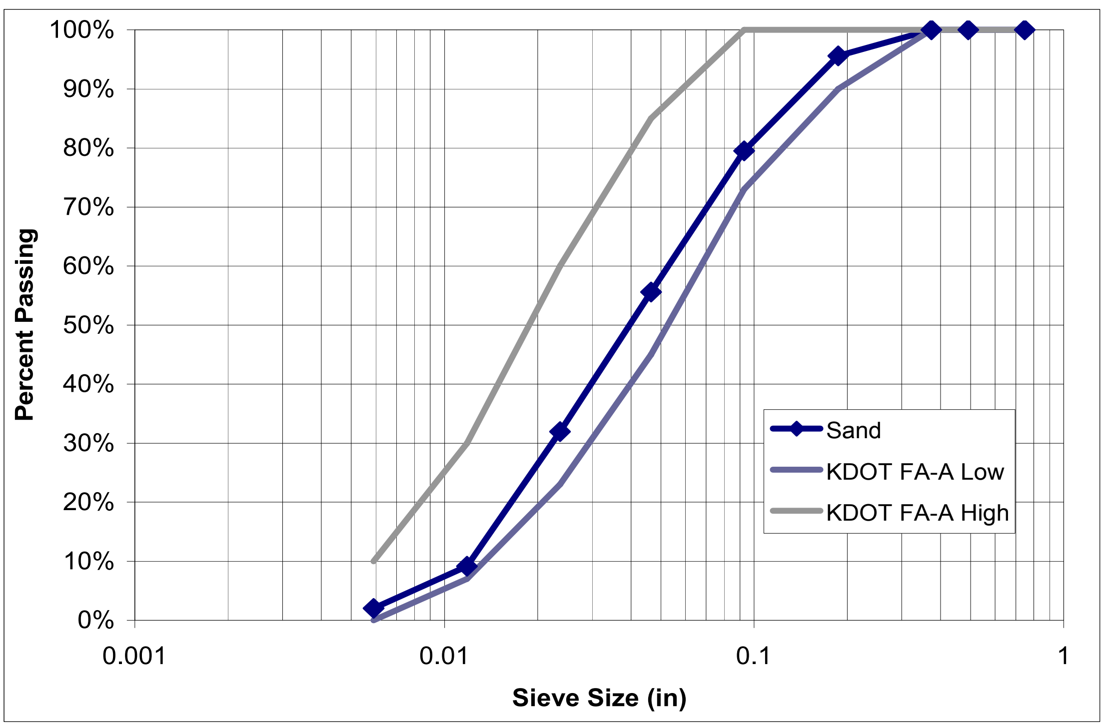

Figure 4.9. Gradation curve of normal-weight sand.

Concrete Mix Design Process

While creating trial concrete mixtures, several mix design variables were altered such as water-to-cement ratio, coarse-to-fine aggregate ratio, and cement content. Each of these variables can have a direct effect on several concrete properties, including a significant effect on strength and workability. For these preliminary lightweight mixtures, an air-entraining admixture was included, since the desired air content was between 5-8%. However, exact air content of each trial mixture was not determined using the volumetric method. Instead, an approximate air content value was calculated using the gravimetric method. The gravimetric method is not exact because it relies on calculations of weight, volume, and specific gravity of the materials in the concrete mixture. Specific gravity of the lightweight aggregate can vary significantly due to the high absorption of the material, resulting in an inaccurate gravimetric air content calculation. However, using this method allowed more mix design variables to be evaluated in a time-efficient manner. In addition, during these preliminary mixtures, correct dosage rate for the air-entraining admixture had not yet been experimentally determined, and consequently several of the gravimetric air contents were not within the specified 5-8% range. Nevertheless, the results obtained between mixes where only one variable was altered are believed to be representative of the concrete properties produced by altering that variable.

Moisture Content and Absorption

Another challenge that needed to be overcome for the preliminary concrete mixes was the high absorption of the lightweight aggregate. At the start of the trial concrete mix designs, absorption rate and maximum absorption values of each of the three aggregate types were still being evaluated and behavior of the material in fresh concrete had not yet been observed. For this reason, all preliminary concrete mixtures were batched with the coarse-lightweight aggregate in the saturated surface dry (SSD) condition. Moisture content of an aggregate sample is composed of the water absorbed within the aggregate and the water located on the surface of the aggregate particle. SSD refers to aggregate that has absorbed water trapped within the aggregate pore structure, but no water on the outer surface of the aggregate particles. This distinction is important because water located on the surface of the aggregate will become part of the mixing water when the concrete is batched. This additional water would significantly affect the water-to-cement ratio if not properly accounted for in the mix design. However, water absorbed within the aggregate is not readily available to cement particles for hydration.

Therefore, this water does not affect the water-to-cement ratio and instead creates a positive internal curing effect, as discussed in Chapter 6.

For this project, the lightweight aggregate was soaked in water for a period of one to seven days, with most of the preliminary mixes having aggregate soaked for one to three days. Prior to batching, the water was strained and the aggregate rolled or rubbed in towels until the glistening sheen of water on the surface of the aggregate could no longer be detected. Pictures of the New Market aggregate in the wet condition, right after the water had been strained, and in the SSD condition, are shown in Figure 4.10 and Figure 4.12, respectively. A picture of the SSD drying process is shown in Figure 4.11. Batching concrete with the aggregate in the SSD condition allowed the absorption and moisture content values to be equal within the absolute volume, mix-design worksheet since no surface water was present to add to the batching water.

Figure 4.10. New Market aggregate in the wet condition.

Figure 4.11. Drying wet aggregate to the saturated surface dry (SSD) condition.

Figure 4.12. New Market aggregate in the SSD condition.

Water-to-Cement Ratio

The first mix design variable evaluated was the water-to-cement ratio. For these trial mix designs, the KDOT minimum cement content of 639 pcy was used (KDOT Section 402). The coarse-to-fine aggregate ratios for each lightweight aggregate type were based on ratios obtained from the Buildex and Stalite companies for similar concrete mix designs. The water-to-cement ratio was expected to affect the workability of the fresh concrete and the compressive strength. In addition, the extra water of the higher water-to-cement ratios could cause higher shrinkage. The KDOT maximum water-to-cement ratio of 0.44 was evaluated along with ratios of 0.42, 0.40, and 0.38 (KDOT Section 402). Slump and compressive strength results are given in Table 4.2 for each aggregate type. Effects varying the water-to-cement ratio can also be seen in Figure 4.13.

Table 4.2. Results of varying the water-to-cement ratio.

Water to Cement Ratio	Marquette Slump (in.)	Marquette 7-Day Compressive Strength (psi)	New Market Slump (in.)	New Market 7-Day Compressive Strength (psi)	Stalite Slump (in.)	Stalite 7-Day Compressive Strength (psi)
0.38	2.25	3210	3.25	3430	2.5	3040
0.40	3.5	3140	3.75	3100	6.25	2910
0.42	5.75	2720	7	2580	7.5	2120
0.44	7.5	1950	7	2180	8	1860

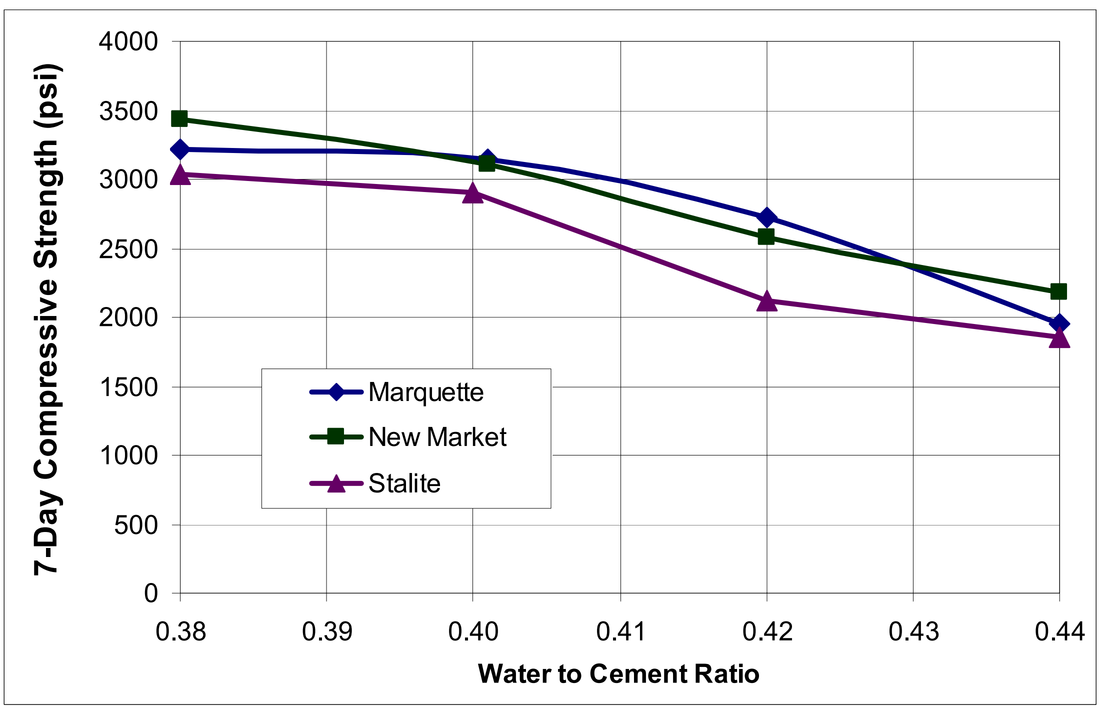

Figure 4.13. Effect of varying water-to-cement ratio on compressive strength.

As expected, compressive strength increased as the water-to-cement ratio decreased for each type of lightweight aggregate. Slump results were also predictable as slump values decreased with lower water-to-cement ratios. At this stage, with the trial mix designs, the 0.38 water-to-cement ratio seemed a bit extreme; so, the 0.40, 0.42, and 0.44 water-to-cement ratios were evaluated for varying coarse-to-fine aggregate ratios.

Coarse-to-Fine Aggregate Ratio

The coarse-to-fine aggregate ratio in lightweight concrete has a direct effect on the mixed-aggregate gradation and concrete unit weight. It will also affect compressive strength. For this project, only sand-lightweight concrete mixtures were considered, meaning that normal-weight sand was used in every mix. All lightweight concrete refers to concrete that contains both lightweight coarse and fine aggregate. If lower unit weight was required, then all lightweight concrete would need to be evaluated. However, for this project, unit weight was not the primary goal, and using sand-lightweight concrete mixtures was adequate to gain a better understanding of the lightweight concrete material and its properties.

Buildex and Stalite manufacturers had supplied sample concrete mix designs that used coarse-to-fine aggregate ratios of 40% coarse, 60% fine for the Marquette aggregate; 46% coarse, 54% fine for the New Market aggregate; and 50% coarse, 50% fine for the Stalite aggregate. These aggregate ratios were used in the varying water-to-cement ratio mixtures shown in Table 4.2. For this stage in the project, each type of lightweight aggregate was tested with coarse-to-fine aggregate ratios of 40% coarse, 60% fine; 50% coarse, 50% fine; and 60% coarse, 40% fine. Each of these coarse-to-fine aggregate ratios was evaluated at water-to-cement ratios of 0.40, 0.42, and 0.44, since the optimum water-to-cement ratio had not yet been decided. Cement content was 639 pcy, as in the varying water-to-cement ratio mixes discussed above. Assuming a 6.5% design air content, the effect of varying the coarse-to-fine aggregate ratio on the unit weight of the concrete is shown in Table 4.3 for each type of lightweight aggregate.

Table 4.3. Effect of varying coarse-to-fine aggregate ratio on concrete unit weight.

Aggregate Ratio	Unit Weight (pcf) Marquette	New Market	Stalite
40% coarse, 60% fine	122.6	124.0	124.2
50% coarse, 50% fine	117.6	119.3	119.5
60% coarse, 40% fine	112.6	114.6	114.8

As shown, design unit weight of concrete increases as the percentage of lightweight coarse aggregate is reduced. In addition, mixed-aggregate gradation is also notably affected by varying the coarse-to-fine aggregate ratio. Gradation curves for each coarse-to-fine aggregate ratio are shown compared to the KDOT MA-2 mixed-aggregate specification limits (KDOT Section 1102). The Marquette aggregate is shown in Figure 4.14, the New Market aggregate is shown in Figure 4.15, and the Stalite aggregate is shown in Figure 4.16.

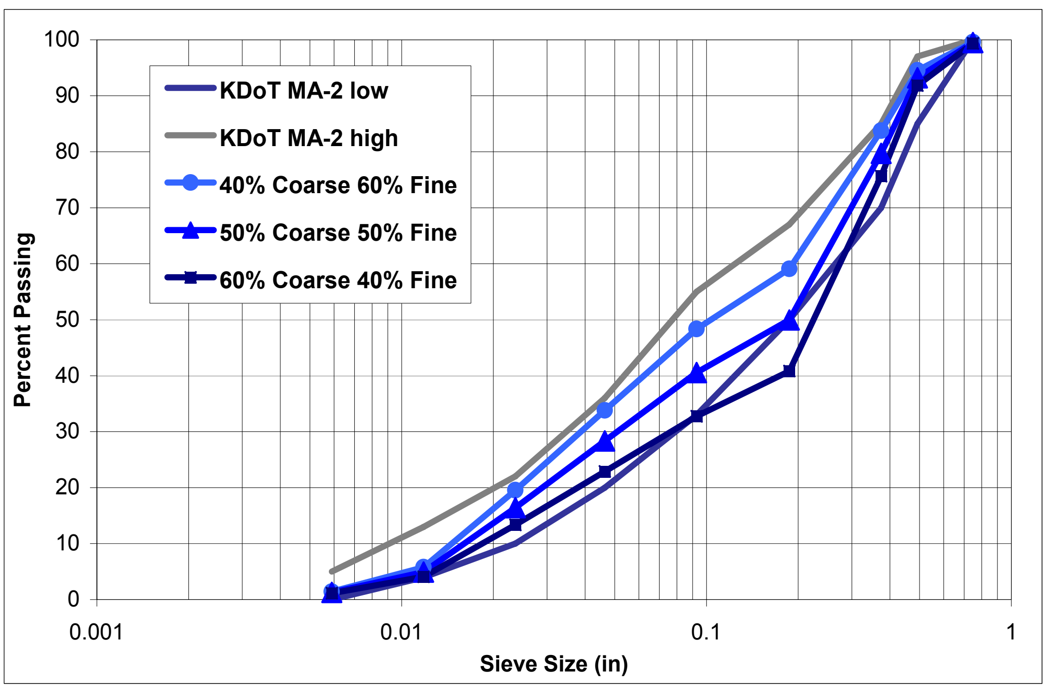

Figure 4.14. Marquette mixed-aggregate gradation for varying aggregate ratios.

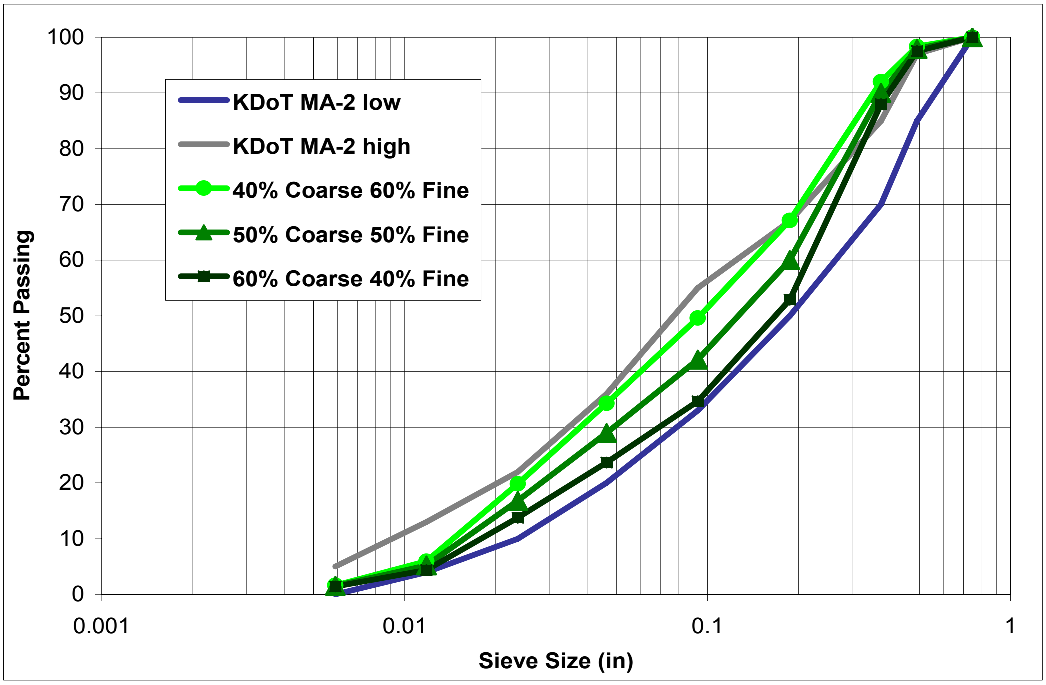

Figure 4.15. New Market mixed-aggregate gradation for varying aggregate ratios.

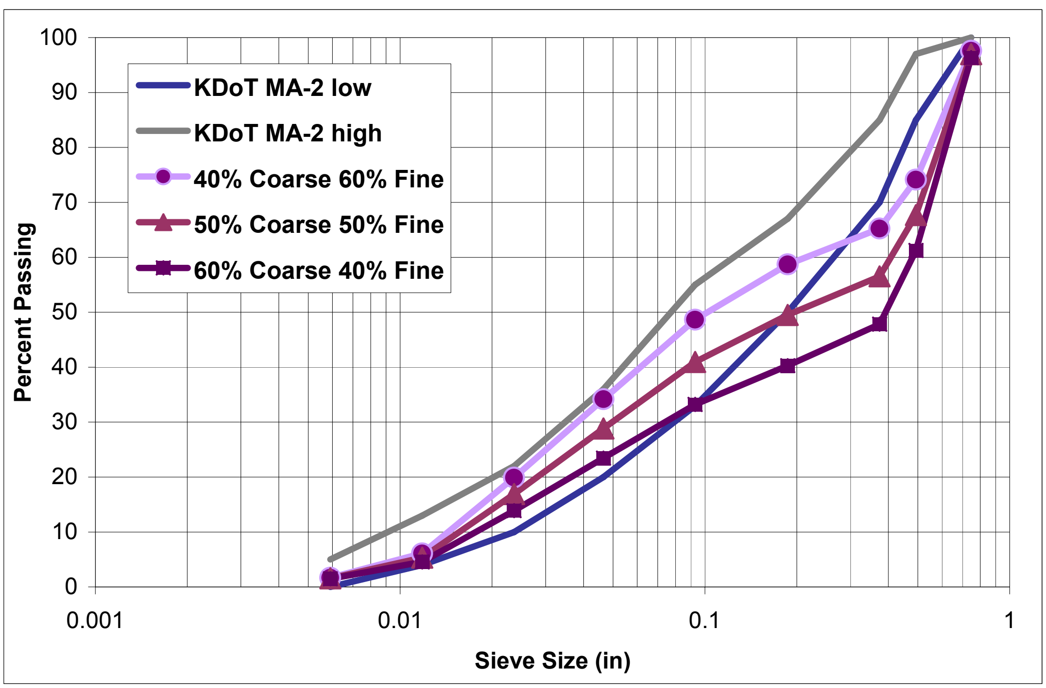

Figure 4.16. Stalite mixed-aggregate gradation for varying aggregate ratios.

As shown in the graphs, none of the coarse-to-fine aggregate ratios fall perfectly within KDOT MA-2 specification limits for New Market and Stalite aggregates, and only the 40% coarse, 60% fine aggregate ratio falls within specification limits for the

Marquette aggregate. For aggregate ratios not within the specification limits, a different fine-aggregate source, with a different gradation, would need to be used or typical gradation of the lightweight aggregate would need to be altered so that the gradations would be in compliance with KDOT limits. However, for this project, the best-fit gradation was chosen to be used for the optimized mix design. The best-fit mixed-aggregate gradation for all three lightweight aggregates is the 40% coarse, 60% fine aggregate ratio. Indeed, this ratio is very close to fitting in the MA-2 range for the New Market and Stalite aggregates, and the Marquette aggregate does fall within the specified limits. A graph of the seven-day compressive strength for each aggregate ratio and water-to-cement ratio is shown in Figure 4.17, Figure 4.18, and Figure 4.19 for the Marquette, New Market, and Stalite aggregates, respectively. As shown, the general trend for each lightweight aggregate is that compressive strength is mostly dependent on the water-to-cement ratio; however, the effect of gradation on compressive strength can also be seen. Lowest strengths are obtained from the 60% coarse, 40% fine aggregate ratios which are the furthest out of the gradation specification limits for each aggregate type. Strength results for the 50% coarse, 50% fine, and 40% coarse, 60% fine aggregate ratios are somewhat inconsistent; but the strengths are comparable and the discrepancy can be attributed to other factors such as air content. The coarse-to-fine aggregate ratio chosen to be used for the optimized mix design was 40% coarse, 60% fine since this ratio best fit KDOT MA-2 specification limits.

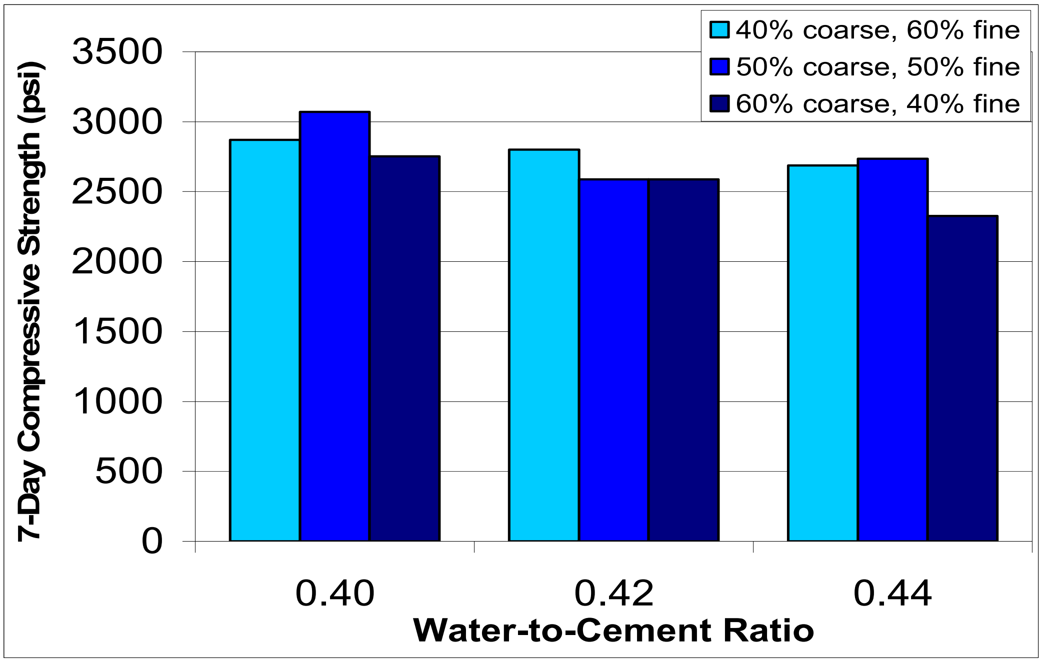

Figure 4.17. Marquette seven-day compressive strength for varying aggregate ratios.

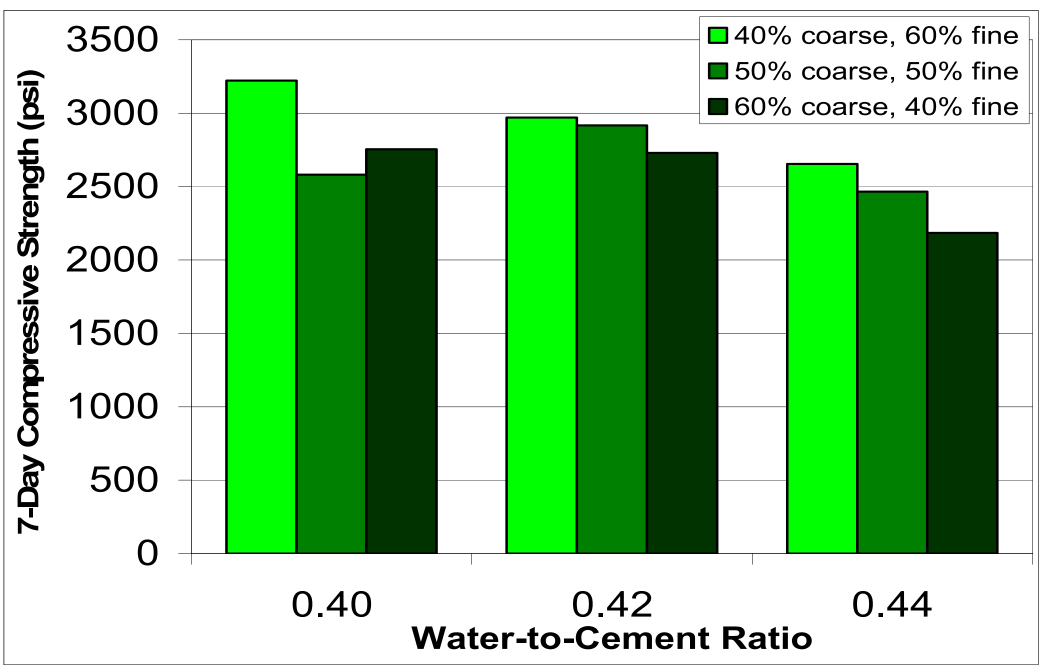

Figure 4.18. New Market seven-day compressive strength for varying aggregate ratios.

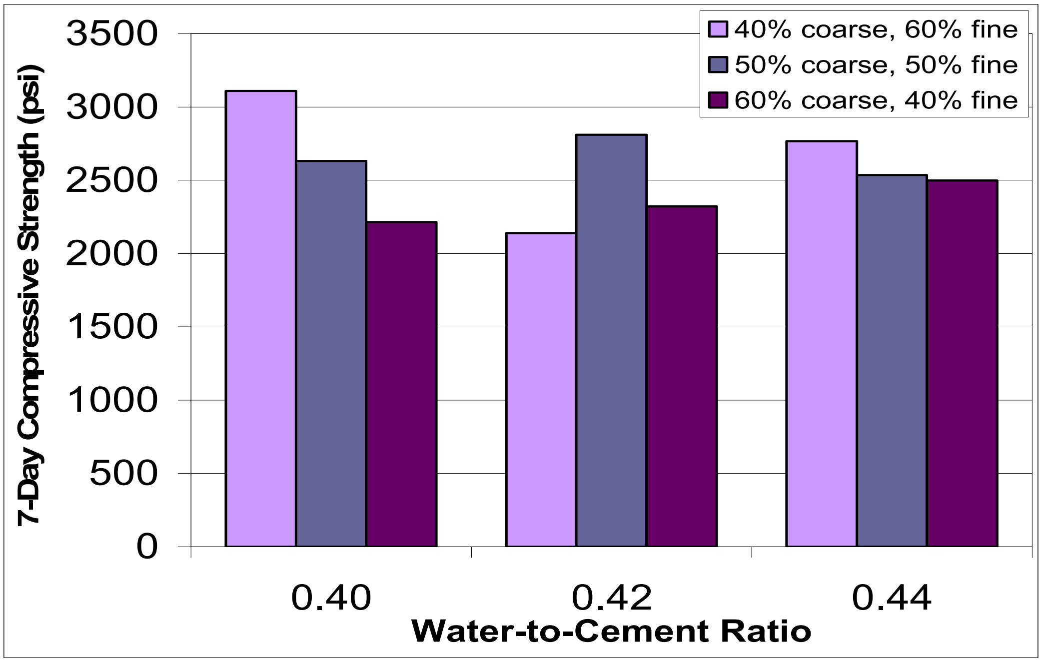

Figure 4.19. Stalite seven-day compressive strength for varying aggregate ratios.

Cement Content

The next mix design variable to be tested was cement content. KDOT specifies a minimum cement content of 639 pcy for grade 4.0 air-entrained concrete for structures (KDOT Section 402). A 28-day compressive strength of 4000 psi was desired for the bridge deck concrete, requiring a laboratory compressive strength of 5200 psi (ACI Committee 318, 2004). Cement content directly affects compressive strength of the concrete. At this stage in the project, higher strengths were needed, so the cement content needed to be increased. However, there was concern that high cement contents would lead to increased shrinkage. Cement is also one of the more expensive components of concrete. Therefore, cement contents of 700 and 750 pcy were tested, while keeping in mind shrinkage and economical concerns. A cement content of 725 pcy was also tested for the Marquette aggregate. The optimized coarse-to-fine aggregate ratio of 40% coarse, 60% fine was used at water-to-cement ratio of 0.40 for all three types of lightweight aggregate. The compressive strength seven-day and 28-day results for all three lightweight aggregate types are shown in Figure 4.20 and Figure 4.21, respectively.

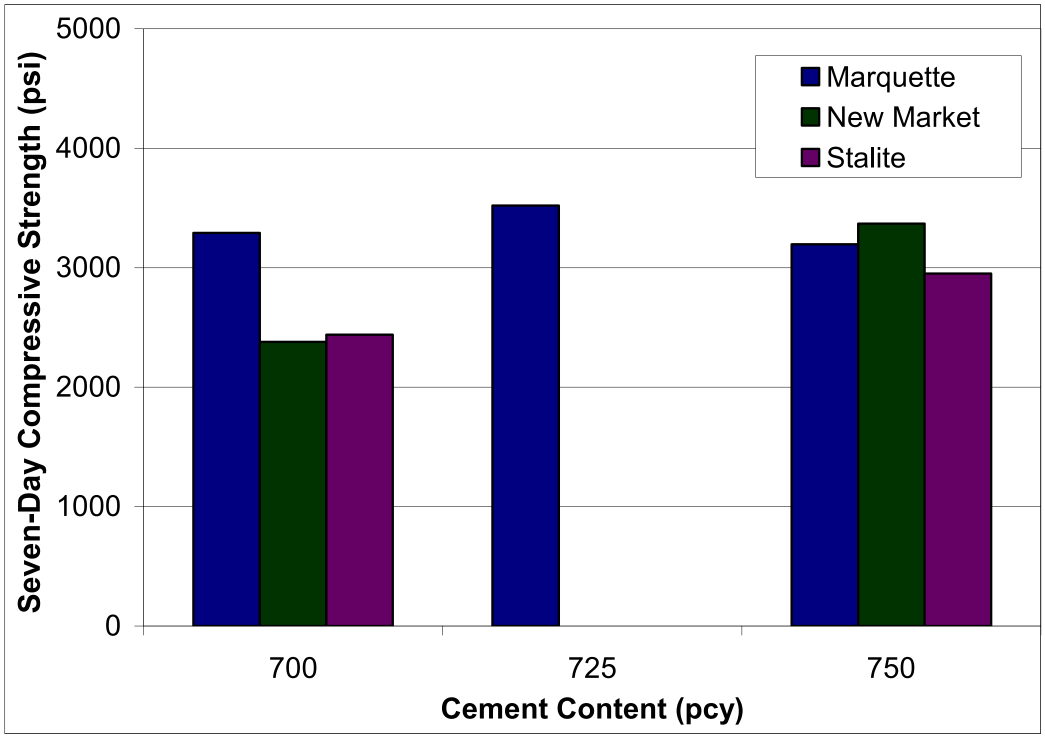

Figure 4.20. Seven-day compressive strength with varying cement contents.

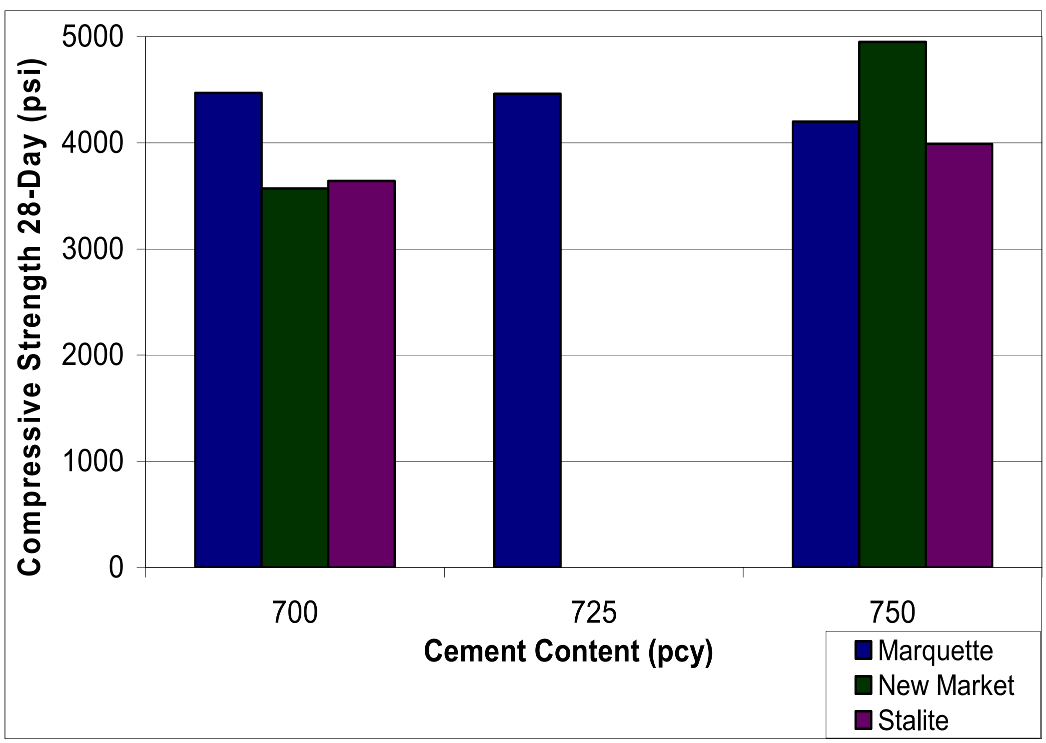

Figure 4.21. Compressive strength, 28-Day with varying cement contents.

Air-Entraining Admixture Dosage

The final variable altered in the bridge deck concrete mixtures was the dosage rate of the air-entraining admixture. The admixture used throughout the project was Daravair 1000 from W. R. Grace Admixtures. KDOT specifies an air content between 5-8% for bridge deck concrete (KDOT Section 402). Up to this point in the project, air content was primarily measured using gravimetric air content. This method can result in inaccurate air contents since the specific gravity of the lightweight aggregate varies and the gravimetric air content calculation relies on the specific gravities of the concrete constituents. Therefore, from this point forward in the project, the rollometer was used. The air-entraining dosage rate was altered for water-to-cement ratios of 0.40 and 0.38 for each of the three types of lightweight aggregate until the proper air content was achieved. Results showed that the air-entraining dosage rate required for 0.75 ft^3 batch was 0.59 oz./100 lbs. cement. This dosage rate consistently produced air contents between 5-8% for all three types of lightweight aggregate. However, it should be noted that this dosage rate cannot be directly scaled up for larger batch sizes, and instead a lower dosage rate is needed for larger batch sizes.

Optimized Bridge Deck Concrete Mixtures

Optimized bridge deck concrete mixtures needed to have a 28-day compressive strength of 4000 psi, requiring a laboratory 28-day compressive strength of 5200 psi. The KDOT maximum water-to-cement ratio, minimum cement content, and air content specifications needed to be met. In addition, unit weight and workability of the concrete mixture needed to be considered. With all of these requirements and considerations, it was decided that the optimized bridge deck mixtures for each of the three types of lightweight aggregate would have a 0.38 water-to-cement ratio; a 40% coarse, 60% fine aggregate ratio; and a 725 pcy cement content. Air-entraining admixture was used at a dosage rate to obtain the required 5-8% air content. Concrete mixes with these variables proved to have adequate compressive strength and workability. Furthermore, this coarse-to-fine aggregate ratio most closely matched KDOT MA-2 gradation limits. However, unit weight of concrete produced with these variables is over 120 pcf for all three types of lightweight aggregate. Lightweight concrete is defined as having a unit weight of 115 pcf or less, with some project specifications allowing densities up to 120 pcf (ACI Committee 213, 1999). These optimized bridge deck mixtures do not meet the lightweight concrete unit weight requirement. Nonetheless, it was decided by KDOT personnel that meeting the KDOT gradation specification was more important than obtaining a lower density concrete. The aggregate gradation would likely have a greater effect on the other material properties being evaluated in the project such as modulus of elasticity, tensile strength, freeze-thaw resistance, permeability, and shrinkage. Therefore, the 40% coarse, 60% fine aggregate ratio continued to be used for the optimized bridge deck mixes. Results of the optimized bridge deck mixtures are shown in Table 4.4 for all three types of lightweight aggregate. As shown, all three of the optimized lightweight bridge deck concrete mixtures satisfy the preliminary design goals.

Table 4.4. Optimized bridge deck mixture results.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)	Compressive Strength 28-day (psi)
Marquette	6.0	121.9	5.5	5410
New Market	7.5	124.3	4.75	6210
Stalite	6.25	122.7	6.25	5220

Lightweight Bridge Deck Mixtures

Lightweight concrete is defined as having a unit weight equal to or less than 115 pcf, with some projects allowing densities up to 120 pcf. Optimized bridge deck concrete mixtures for all three types of lightweight aggregate had unit weights over 120 pcf. Therefore, it was decided to test the basic properties of concrete mixtures similar to the optimized bridge deck mixtures, but with design unit weights of 115 pcf. To accomplish this lower unit weight, the coarse-to-fine aggregate ratio was altered until a design unit weight of 115 pcf was achieved for each type of lightweight aggregate. The specific gravity of the Marquette aggregate was 1.44, whereas the New Market and Stalite aggregates had a specific gravity of 1.52; therefore, different coarse-to-fine aggregate ratios resulted. The Marquette had a 57.2% coarse, 42.8% fine aggregate ratio and the New Market and Stalite aggregates had a 68.4% coarse, 31.6% fine aggregate ratio. The effect of altering the coarse-to-fine aggregate ratio on the KDOT MA-2, mixed-aggregate gradation is shown in Figure 4.22, Figure 4.23, and Figure 4.24 for each type of lightweight aggregate. As shown, gradation of these mixes is not within MA-2 limits; however, this restriction was ignored for these particular mixtures to determine if adequate strength and unit weight could be achieved. To further facilitate lowering the design unit weight, a cement content of 675 pcy was used. In addition, to offset the expected decrease in strength, the water-to-cement ratio was lowered to 0.36 for the typically lower strength Marquette and Stalite aggregates and to 0.37 for the generally higher strength New Market aggregate.

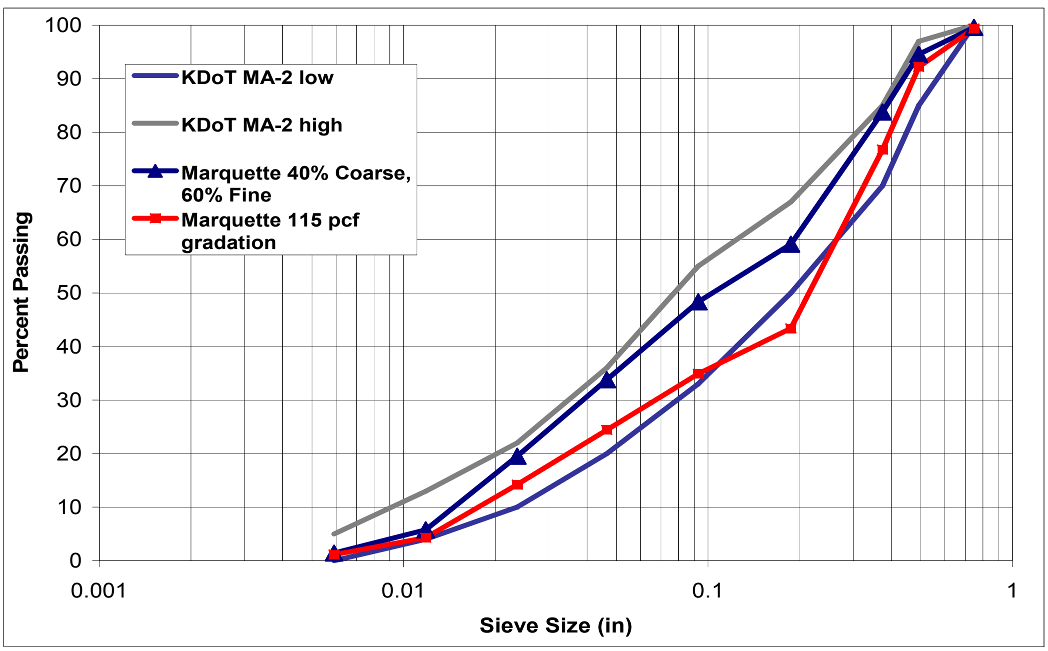

Figure 4.22. Marquette aggregate gradation for optimum bridge deck and 115 pcf concrete mixtures.

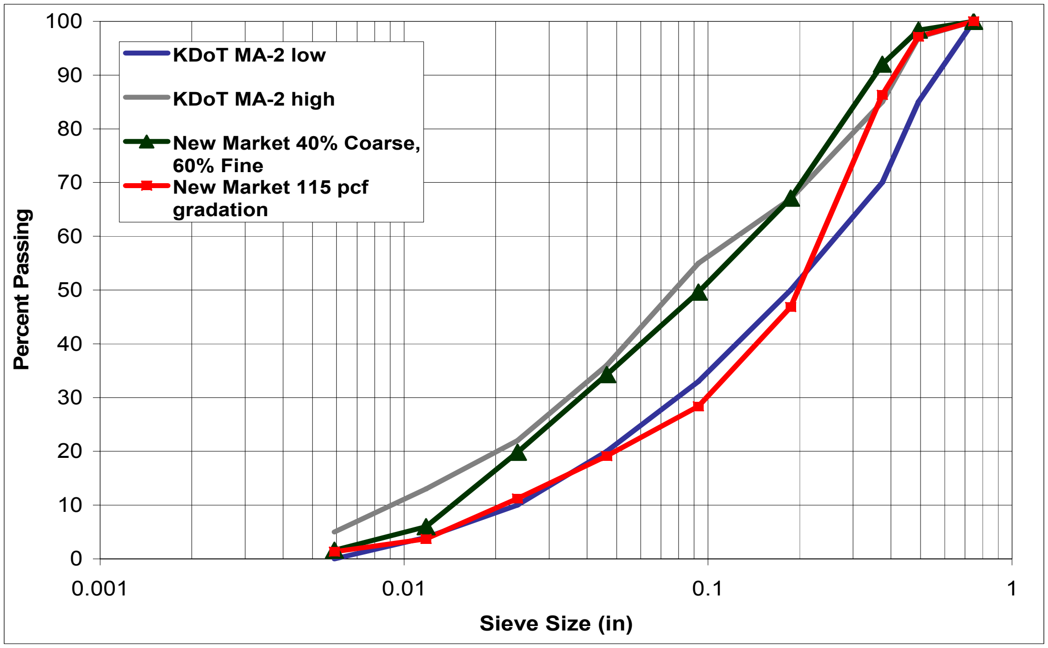

Figure 4.23. New Market aggregate gradation for optimum bridge deck and 115 pcf concrete mixtures.

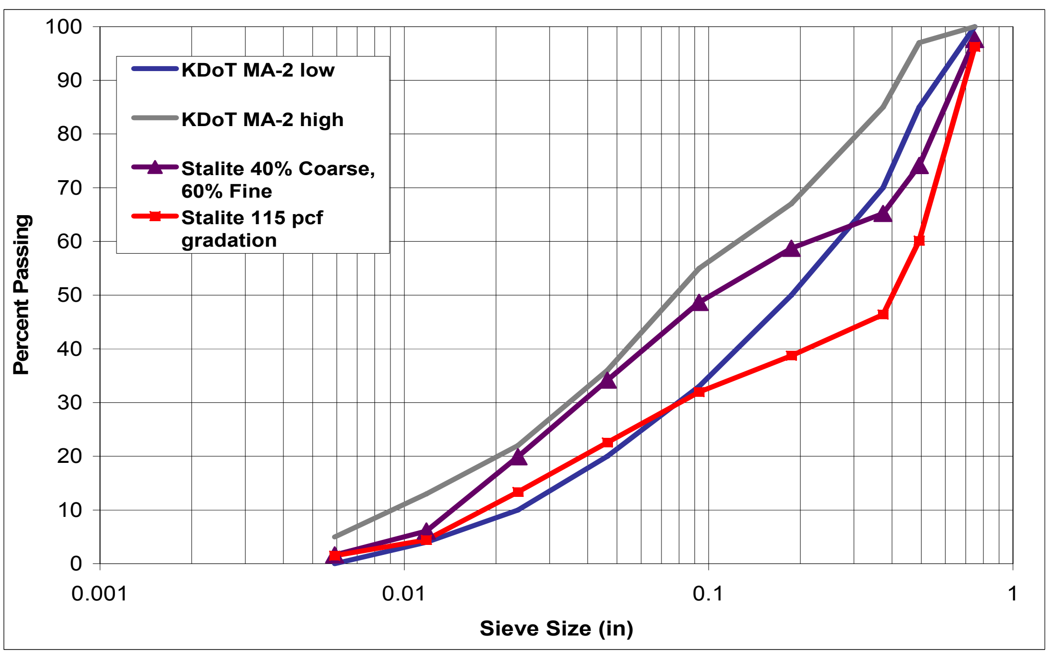

Figure 4.24. Stalite aggregate gradation for optimum bridge deck and 115 pcf concrete mixtures.

Results from these 115 pcf design unit weight mixtures showed that adequate compressive strength could be obtained from batch unit weights less than 120 pcf using a cement content of 675 pcy and water-to-cement ratios of 0.36 and 0.37 for the different aggregate types. Batch unit weights obtained for each type of lightweight aggregate and other results are given in Table 4.5. These mixes demonstrated adequate compressive strength and lower unit weight; however, it should be noted that altering the coarse-to-fine aggregate ratios not only affected the gradation, but also workability of the fresh concrete. It was observed that the plastic concrete was noticeably less workable and harder to finish, especially for the Stalite aggregate which was pushed the furthest out of the gradation limits.

Table 4.5. Results for 115 pcf design unit weight concrete mixtures.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)	Compressive Strength 28-day (psi)
Marquette	4.5	118.6	1.5	5820
New Market	6.0	117.0	1.0	5860
Stalite	5.0	117.7	2.5	6310

Lightweight Prestressed Concrete Mixtures

Several preliminary lightweight prestressed concrete mixtures were created and tested. Main design goals for the prestressed concrete mixes included a 16-hour compressive strength of 5000 psi and a unit weight under 120 pcf. Other goals were to obtain reasonable workability and meet KDOT specifications of a minimum cement content of 639 pcy, a maximum water-to-cement ratio of 0.35, and an air content between 5% and 8% (KDOT Section 402).

Materials

To achieve the desired 16-hour compressive strength of 5000 psi, high early strength, Type III cement was used. Two different shipments of cement were used throughout the project, the first obtained from Ash Grove Cement Company and the second from Lonestar Cement Company. No noticeable difference was observed from using the two different sources of cement. The fine aggregate used was the same as for the bridge deck mixtures. Admixtures consisted of an air-entraining admixture, Daravair 1000, also used in the bridge deck mixtures, and two types of superplasticizer admixtures. Daravair 1000 meets AASHTO M154 for air-entraining admixture. The first superplasticizer tried was Daracem 100, the second ADVA Cast 530, both meet AASHTO M194 specification for chemical admixtures for concrete. All admixtures were obtained from W.R. Grace Admixtures.

Concrete Mixture Design Process

Experience gained from obtaining optimized bridge deck concrete mixtures was applied to design the lightweight prestressed concrete mixtures. A cement content of 725 pcy was chosen to be used along with a decreased water-to-cement ratio of 0.34 to achieve the needed 5000 psi, 16-hour compressive strength. Coarse-to-fine aggregate ratios were kept at 40% coarse, 60% fine for most of the preliminary mixtures, then altered until a design unit weight of 118 pcf was achieved. This design unit weight resulted in aggregate ratios of 52% coarse, 48% fine for the Marquette aggregate, and 56% coarse, 44% fine for the New Market and Stalite aggregates.

Admixture Dosage Rate

To achieve reasonable workability with a 0.34 water-to-cement ratio, a superplasticizer admixture was required in addition to the air-entraining admixture. The first superplasticizer tried was Daracem 100, classified as a Type F and Type G superplasticizer according to AASHTO M194. In addition to being a high-range water reducer, Daracem 100 was also chosen for its ability to provide an increased time span of high slump. The air-entraining admixture, Daravair 1000, is compatible with Daracem 100 according to the manufacturer. However, significantly sporadic and unrepeatable air contents were obtained when using both the Daravair 1000 air-entraining admixture and the Daracem 100 superplasticizer in several of the preliminary lightweight prestressed concrete mixtures. Several different combinations of dosage rates were experimented with for both the Daravair 1000 and Daracem 100 admixtures. The problem arose with the fresh concrete air content not being within the specified 5-8% limits. The effect of using the superplasticizer admixture seemed to exponentiate the effect of the air-entraining admixture, producing much higher air contents at normal dosage rates. For this reason, the air-entraining dosage rate was used at a lower dosage rate than that recommended by the manufacturer. Some concrete mixtures, made with these two admixtures, did meet air content limits. However, these same mixtures proved to not be repeatable. For this reason, the superplasticizer admixture was changed to ADVA Cast 530.

ADVA Cast 530 is a Type F, polycarboxylate-based superplasticizer typically used for prestressed applications. At low dosage rates it can be used for conventional concrete and at high dosage rates it can be used for self-consolidating concrete. Several preliminary concrete mixtures were also developed with ADVA Cast 530 and the air-entraining admixture, Daravair 1000. Dosage rates of these two admixtures were adjusted until air contents between 5% and 8% were achieved for all three types of lightweight aggregate. These concrete mixtures did prove to be repeatable for batches of the same size. Successful dosage rates used for smaller batches in this project were 0.15 oz./100 lb. cement for the air-entraining admixture, and 6.5 to 6.9 oz./100 lb. cement for the different types of lightweight aggregate. However, it should be noted that the admixture dosage rates cannot be directly scaled up for larger batch sizes, and further dosage rate adjustments should be made. A comprehensive list of lightweight prestressed concrete mixtures batched with both the Daracem 100 and ADVA Cast 530 superplasticizer admixtures is shown in Appendix A -Concrete Mixture Summary.

Optimized Prestressed Concrete Mixtures

An adequate 16-hour compressive strength of 5000 psi was obtained for each type of lightweight aggregate using 725 pcy of Type III cement and a 0.34 water-to-cement ratio. In order to obtain more realistic 16-hour compressive strength values, cylinders were placed in a heated water tank to simulate heat produced due to cement hydration reaction by a larger mass of concrete. Cylinders were placed in the tank after initial set, and water surrounded the cylinders up to about 0.5 in. below the top of the cylinder. The water was left at room temperature until approximately three hours after the cylinders were placed in the tank, then the water was heated to 150°F for a period of five hours. Several prestressed concrete mixtures were tested for 16-hour heated and unheated compressive strength. The 16-hour compressive strength was noticeably higher for all three types of lightweight aggregate when using this procedure. A table of 16-hour heated and unheated compressive strengths is shown in Table 4.6. It is believed that the compressive strength of the heated cylinders more accurately represents the compressive strength achieved in a larger mass of concrete.

Table 4.6. Prestressed concrete heated and unheated 16-hour compressive strength.

	Compressive Strength Unheated 16-hour (psi)	Compressive Strength Heated 16-hour (psi)
Marquette	4670	5470
New Market	5020	5630
Stalite	5710	6460

Air-entraining and superplasticizer admixtures were experimented with until air content was within the KDOT specified 5-8% limits and a reasonable workability was achieved. Most preliminary prestressed concrete mixtures were designed with the 40% coarse, 60% fine aggregate ratio, as used in the optimized bridge deck mixtures. However, a unit weight lower than 120 pcf was desired. Therefore, final optimized prestressed mixtures were created with altered coarse-to-fine aggregate ratios to achieve the design unit weight of 118 pcf. The 118 pcf design unit weight was chosen to limit the negative effect on the mixed-aggregate gradation required to achieve a 115 pcf design unit weight, while still reaching the defined unit weight limits for structural lightweight concrete of less than 120 pcf. A design unit weight of 120 pcf was not used in case the exact 6.5% design air content was not achieved or the specific gravity of the lightweight aggregate was slightly heavier than accounted for. Results for the successful, optimized lightweight prestressed concrete mixtures are shown in Table 4.7.

Table 4.7. Results for optimized lightweight prestressed concrete mixtures.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)	Compressive Strength 16-hour (psi)
Marquette	8.0	117.2	7	5480
New Market	6.0	115.0	9	5900
Stalite	7.0	116.6	9	5160

In addition to the basic prestressed concrete mixture design goals of compressive strength, air content, and reasonable workability, prestressed concrete mixtures with slumps of 3 in. and 9 in. were desired. These mixtures were needed to test the effects of a high-slump versus low-slump concrete on strand bond for another part of the project. To accomplish the 3-in. and 9-in. slumps, batches were composed of identical cement content, water-to-cement ratio, and coarse-to-fine aggregate ratio to the optimized prestressed concrete mixtures, and the admixture dosage rates were altered until the proper air content and slump was achieved. In many cases, slump was allowed to be greater than the desired 3-in. or 9-in. requirement, and the concrete continued to be mixed until slump loss occurred and desired slump was achieved.

CHAPTER 5 - Experimental Design and Setup

This chapter describes the experimental test setup and test specimens for several concrete properties evaluated during this project. Concrete properties studied included lightweight-aggregate moisture content and absorption, and concrete compressive strength, tensile strength, modulus of elasticity, freeze-thaw resistance, permeability, alkali-silica reactivity, and shrinkage. Several of the concrete property tests were conducted for both the optimized bridge deck and optimized prestressed lightweight concrete mixtures.

Large-Batch Moisture Content

Lightweight aggregate absorbs significantly more water than normal-weight aggregate. Approximate maximum absorption values for the lightweight aggregates studied in this project are 26% for Marquette, 29% for New Market, and 8% for Stalite. Details of absorption rates for each type of lightweight aggregate are discussed in Chapter 3 along with 24-hour and 90-day absorption-rate graphs shown in Figure 3.14 and Figure 3.15, respectively. As discussed in Chapter 3, absorption and moisture content of the aggregate needs to be accurately accounted for in the concrete mix design when using the absolute volume method. For all preliminary concrete batches created for the bridge deck and prestressed mixtures, the lightweight aggregate was batched in the saturated surface dry (SSD) condition. In the SSD condition, all of the water was absorbed into the internal cellular structure of the aggregate and the surface water was removed by rubbing the aggregate in towels until the sheen of the surface water disappeared. Using this method allowed small batches of concrete to be created and tested while the absorption rate of the three types of lightweight aggregate was still being studied. In the SSD condition, the water-to-cement ratio of the concrete mixture was not altered since the free surface water was removed prior to batching. Therefore, in the SSD condition, the assumption can be made that the moisture content of lightweight aggregate is only composed of the absorbed water and will not alter the water-to-cement ratio of the concrete mixture by either adding additional surface water to the mixture or absorbing significant amounts of water out of the mixture. This method proved to be successful and repeatable for several preliminary bridge deck and prestressed concrete mixtures. However, batching large amounts of concrete with the aggregate in the SSD condition would not be feasible due to time constraints. Therefore, a method was developed to batch concrete using pre-soaked but strained lightweight aggregate, accounting for the moisture content and absorption in the concrete mix design.

Since lightweight aggregate continues to absorb water over a significant time period, moisture content and absorption values were determined at 1, 3, 7, 10, 14, 21, 28, and 60 days for each type of lightweight aggregate. For this test, the aggregate was submerged in five-gallon buckets until the specified time period. The water was then strained out of the bucket and a sample of at least 2000g of the wet aggregate was immediately placed in a pan and weighed. Next, some of the remaining aggregate was dried until the SSD condition was reached, and this sample of at least 2000g was placed in a pan and weighed. Moisture content of each of these samples was then determined according to AASHTO T255. The aggregate in the wet condition was considered to represent the total moisture content of the strained wet aggregate. The aggregate in the SSD condition was considered to represent the absorbed water located within the internal cellular structure of the lightweight aggregate. The difference between these two values is then the amount of additional free water available in the concrete mixture when batched. For concrete mixes with lightweight aggregate soaked for time periods not measured, moisture content and absorption values were interpolated from surrounding values.

For large concrete batches, the lightweight aggregate was soaked in 55-gallon barrels and holes were drilled in a barrel lid to allow the water to be strained from the aggregate. A mesh screen was also placed under the lid to prevent smaller aggregate particles from going through the holes in the barrel lid. The lid was tightened and the barrel was then inverted, using a forklift and barrel mover, until the water drained. The same moisture content and absorption values were used for small and large batches. A picture of the water being strained from the lightweight aggregate for both small and large batches is shown in Figure 5.1 and Figure 5.2.

Figure 5.1. Water being strained for small concrete batch.

Figure 5.2. Water being strained from lightweight aggregate for large concrete batch.

Compressive Strength

Compressive strength was determined for both the optimized bridge deck and prestressed concrete mixtures for each type of lightweight aggregate. Tests were conducted according to AASHTO T22. Compressive strength was measured at time periods of 1, 3, 7, 14, 21, and 28 days for the optimized bridge deck mixtures and 16-hours, 3, 7, 14, 21, and 28 days for the optimized prestressed concrete mixtures. All of the compressive strength, tensile strength, and modulus of elasticity specimens were made from the same batch of concrete for each type of lightweight aggregate. For the compressive strength, three 4-in. x 8-in. cylinders, made according to AASHTO T126, were tested for each time period. Care was taken to keep the cylinders moist before testing by covering them with a wet towel after removal from the lime-water storage tank. The diameter of each cylinder was measured twice at mid-height of the cylinder at 90° angles. These values were then averaged and the area of the cylinder was determined from the average diameter value. Each specimen was then tested in a hydraulic compression machine until failure. Sulfur caps were used for the optimized bridge deck concrete mixtures, and neoprene pads and caps were used for the prestressed concrete compressive strength tests. In addition, a canvas wrap was used for safety and to minimize the mess produced by failure of the cylinder. A picture of the compressive strength test setup is shown in Figure 5.3. The compressive strength of each specimen was then calculated as the maximum compressive load divided by the area of the cylinder. Finally, average compressive strength was determined from the compressive strength values of the three individual cylinders tested for a given day.

Figure 5.3. Compressive strength test setup.

It should be noted that for the optimized bridge deck concrete mixtures, the same 4-in. x 8-in. cylinders were used for both the modulus of elasticity and compressive strength tests. This method was chosen so that a smaller and more manageable batch size was required. The process of reusing cylinders for the modulus of elasticity and compressive strength should theoretically not affect the compressive strength results since the cylinders are exposed to less than 40% of their compressive strength capacity to determine the modulus of elasticity. However, experimental compressive strength results obtained are in some cases lower than expected results for similar mixtures. The difference in this case was attributed to the re-use of the cylinders and micro-cracking caused during the modulus of elasticity test. Re-using the optimized bridge deck compressive strength cylinders to determine the modulus of elasticity is also the reason for using sulfur caps on the bridge deck cylinders.

Tensile Strength

Tensile strength was determined for both the optimized bridge deck and optimized prestressed concrete mixtures. To determine tensile strength, the splitting tensile strength of cylindrical concrete specimens test was used according to AASHTO T198. Tensile strength was measured at time periods of 1, 7, 14, 21, and 28 days for the optimized bridge deck mixtures and 16-hours, 3, 7, 14, 21, and 28 days for the optimized prestressed mixtures. All compressive strength, tensile strength, and modulus of elasticity specimens were made from the same batch of concrete for each type of lightweight aggregate. For the tensile strength test, three 6-in. x 12-in. cylinders were tested for each time period. Care was taken to keep the cylinders moist after removal from the lime-water storage tank and before testing by covering the specimens with a wet towel. Cylinder length and diameter at mid-height was measured three times for each specimen and average values for each calculated. Each specimen was loaded in an apparatus as described in AASHTO T198 until failure occurred. A picture of the split tensile test setup is shown in Figure 5.4. Tensile strength capacity was then determined using the equation

$$T = \frac{2P}{\pi l d}$$

where
$$\begin{aligned} T &= \text{tensile strength (psi)} \\ P &= \text{maximum load (lb.)} \\ l &= \text{cylinder length (in.)} \\ d &= \text{cylinder diameter (in.)} \end{aligned}$$

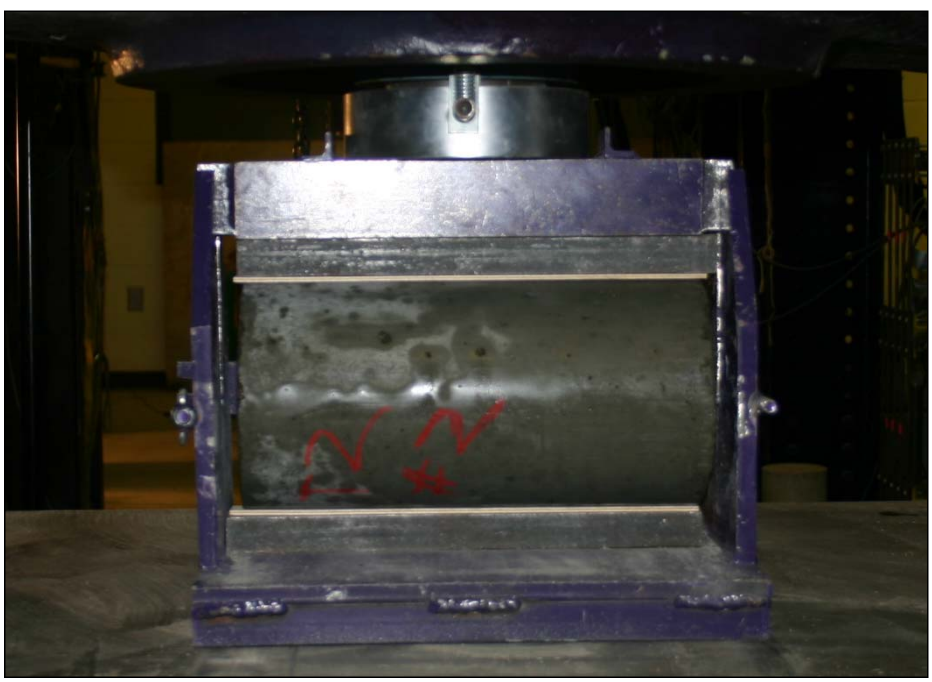
Figure 5.4. Splitting tensile strength test setup.

Modulus of Elasticity

The modulus of elasticity was determined for both the optimized bridge deck and prestressed concrete mixtures according to ASTM C469. Three, 4-in. x 8-in. cylinders were used for each test at time periods of 1, 3, 7, 14, 21, and 28 days for the optimized bridge deck mixtures and 16-hours, 3, 7, 14, 21, and 28 days for the optimized prestressed mixtures. All compressive strength, tensile strength, and modulus of elasticity specimens were made from the same batch of concrete for each type of lightweight aggregate. To determine the modulus of elasticity, sulfur caps were used on both the optimized bridge deck and prestressed concrete mixtures. It was necessary to use sulfur caps instead of the neoprene pads used for the compressive strength tests because otherwise the steel controllers of the neoprene caps would hit the compressometer. The diameter of each cylinder was measured twice at mid-height and the average diameter calculated. The specimens were then placed in the compressometer and loaded using a hydraulic testing machine. The compressometer used was digital, and an MTS system was used to collect load and displacement data several times a second for each load cycle. A picture of the compressometer and test setup is shown in Figure 5.5. The load range did not exceed 40% of the maximum compressive strength, and oftentimes specimens were only loaded to 25-30% of the maximum compressive strength since for the optimized bridge deck mixtures, maximum compressive strength was estimated since the cylinders were re-used for the compressive strength test. Each cylinder was loaded three times, and the modulus of elasticity was not calculated from the data obtained in the first loading. The modulus of elasticity for each cylinder was taken as the average value calculated from the second and third loadings. The chord modulus of elasticity was calculated using the equation

$$E_{chord} = \frac{(S_2 - S_1)}{(\varepsilon_2 - 0.000050)}$$

where

E_{chord} = cord modulus of elasticity
S_1 = stress corresponding to longitudinal strain of 0.000050 (psi)
S_2 = stress corresponding to maximum load (psi)
ε_2 = longitudinal strain produced by S_2

In addition to the chord modulus of elasticity, the modulus of elasticity was also calculated using Excel. In Excel, stress and strain for each data point of the second and third loading cycle was calculated, then graphed up to the maximum load. A trendline, or best-fit line, was then plotted on the graph and the slope of this line was used as an average modulus of elasticity value. This value is referred to as E_{graph} in Chapter 6. For both calculation methods, an average modulus of elasticity for all three cylinders for a given time period was then calculated and taken as the reported modulus of elasticity.

It should be noted that on several of the younger, optimized bridge deck specimens, there was significant scatter in the data points obtained. The cause of this phenomenon is believed to be due to the lower strength of the specimens since Type I cement was used. However, since the modulus was calculated from the slope of the data trend, the calculated modulus of elasticity values were still believed to be representative of the true modulus of elasticity.

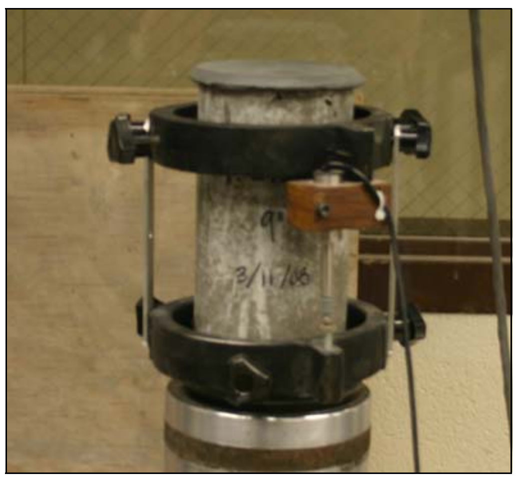

Figure 5.5 Compressometer setup for modulus of elasticity test.

Freeze-Thaw Resistance

The *Resistance of Concrete to Rapid Freezing and Thawing* test was performed according to AASHTO T161 and the KDOT KTMR-22 specification. This test is used to determine the resistance to freezing and thawing cycles of variations of concrete properties, in this case different types of lightweight aggregate. Two standard procedures are Procedure A, which consists of rapid freezing and thawing in water, and Procedure B, which consists of rapid freezing in air and thawing in water. If performance results show the concrete to be relatively unaffected, it can be surmised that the specimens were not critically saturated, or that the concrete was made of sound aggregate, a proper air-void system, and was allowed to mature properly.

For this project, Procedure B was used. Six, 3-in. x 4-in. x 16-in. beams were made for the optimized bridge deck concrete mixtures. Three beams were needed for one test set, so two sets of freeze-thaw beams were made and tested for each type of lightweight aggregate. The same batch of concrete was used to make freeze-thaw resistance, permeability, and alkali-silica reactivity test specimens. The prestressed concrete mixes were not evaluated for freeze-thaw resistance since it was unlikely that prestressed bridge beams would be significantly exposed. The beams were moist cured for 67 days, then cured in air at 50% relative humidity for 21 days. The beams were then submerged in a tempering tank for 24 hours and placed in a freezer for 24 hours. At this point, the freezing and thawing cycles began. Initial mass, length, and fundamental transverse frequency was determined. A freezing and thawing cycle consists of alternately lowering the temperature to 0°F, then raising it to 40°F in not less than two, nor more than five, hours. Specimens were removed from the test apparatus at intervals not exceeding 36 freezing and thawing cycles, and changes in length, mass, and fundamental transverse frequency were measured. Freezing and thawing cycles were continued until the specimens were subjected to 300 cycles. The standard freeze-thaw test continued until specimens had been subjected to 300 cycles or until the relative dynamic modulus of elasticity reached 60% of the initial modulus, whichever occured first. The relative dynamic modulus of elasticity is calculated using the equation

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

where

P_c = relative dynamic modulus of elasticity, after c cyles of freezing and thawing (%)
n = fundamental transverse frequency at 0 cycles of freezing and thawing
n_1 = fundamental transverse frequency after c cycles of freezing and thawing

Next, the durability factor is calculated using the equation

$$DF = PN/M$$

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

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

According to the KDOT 1102, Coarse Aggregates for Concrete specification, acceptable values for resistance to freezing and thawing is a durability factor of 95 or higher and an expansion not greater than 0.025%. Pictures of the freeze-thaw resistance test set-up is shown in Figure 5.6 and Figure 5.7.

Figure 5.6. Freeze-thaw resistance testing apparatus.

Figure 5.7. Freeze-thaw beams in testing tank.

Permeability

The *Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration* test was used according to AASHTO T277 to determine permeability of the optimized bridge deck concrete mixtures. It is unlikely that the prestressed concrete mixtures would be significantly exposed to de-icing salts in the field; therefore, these mixtures were not tested for permeability. This test consisted of monitoring the amount of electrical current passing through 2-in. thick slices of 4-in. diameter cylinders during a six-hour period, taking readings at least every 30 minutes. One end of the specimen was submerged in a sodium-chloride solution, and the other end immersed in a sodium-hydroxide solution, with a potential difference of 60 V maintained across the ends of the specimen. The total charge passed, measured in coulombs, indicated the resistance of the specimen to chloride-ion penetration. For this project, the same batch of concrete was used to make freeze-thaw resistance, permeability, and alkali-silica reactivity test specimens. Six, 4-in. x 8-in. cylinders were made for all three lightweight aggregate types. The cylinders were moist cured for 28 days, then stored in air with 50% relative humidity for an additional 28 days. The test was conducted at a time period of 56 days. The reported value was determined by drawing a smooth curve through the data and then integrating underneath the curve to obtain the coulombs of charge passed during the six-hour test period. Alternatively, automatic data processing equipment can be used to determine the coulomb value.

Factors known to affect chloride-ion penetration are the water-to-cement ratio, presence of polymeric admixtures, sample age, air-void system, aggregate type, degree of consolidation, and type of curing. According to AASHTO T277, the correlation of chloride-ion penetrability based on charge passed in this test is shown in Table 5.1.

Table 5.1. Chloride-ion penetrability based on charge passed.

Charge Passed (Coulombs)	Chloride Ion Penetrability
> 4000	High
> 2000 – 4000	Moderate
> 1000 – 2000	Low
100 – 1000	Very Low
< 100	Negligible

Alkali-Silica Reactivity

The *Wetting and Drying Test of Sand and Sand-Gravel Aggregate for Concrete* was conducted according to the KDOT KTMR-23 procedure to indicate alkali-silica reactivity of the optimized bridge deck lightweight concrete mixtures. The prestressed concrete mixtures were not subjected to this test since it is unlikely that prestressed bridge beams would be exposed to significant amounts of wetting and drying. This test consisted of making six, 3-in. x 4-in. x 16-in. beams and then curing the specimens for seven days in a moist room. The same batch of concrete was used to make freeze-thaw resistance, permeability, and alkali-silica reactivity test specimens for each type of lightweight aggregate. The beams were then stored in air at 50% relative humidity for 21 days. At 28 days, the air-dry mass and length were determined, along with the SSD mass and length after the beams were placed in water for a minimum of one hour. The beams were then stored in water for 48 hours. Next, the three beams to be tested in flexure at 60 days were returned to the moist room for an additional 30 days of curing. At 60 days after casting, the three beams were tested for modulus of rupture according to AASHTO

T 177 to determine flexural capacity. After completing the flexural test, both remaining halves of the beams were broken according to AASHTO T140 to determine compressive strength. The other three beams were subjected to the wetting and drying test procedure beginning 30 days after casting. One cycle of the wetting and drying test procedure consisted of placing the beams in an oven maintained at 128-130°F for eight hours, then removing the beams and submerging them in a water bath maintained at 60-80°F for 15.5 ± 0.5 hours. A picture of the wetting and drying beams in the test oven is shown in Figure 5.8. Each cycle was completed in 24 hours, and the cycle was repeated each consecutive day throughout the 365-day period except for weekend and holidays when the beams remained in the water bath. Length was measured at time periods of 30, 60, 120, 180, 240, 300, and 365 days and the change in length was calculated. After completion of the wetting and drying test cycles at 365 days, the beams were tested for flexural modulus of rupture and then compressive strength, as with the first three beams tested at 60 days.

Requirements for acceptability of the aggregate are based on both the sets of beams tested at 60 days and 365 days. Each set of beams must have a flexural modulus of rupture not less than 550 psi. In addition, the percent length change of the beams exposed to the wetting and drying test cycles must not exceed 0.050% at 180 days, nor 0.070% at 365 days.

Figure 5.8. Alkali-silica reactivity test beams in oven.

Shrinkage

Drying shrinkage and autogenous shrinkage were determined for the optimized bridge deck and optimized prestressed concrete mixes, for all three types of lightweight aggregate. Many factors can affect shrinkage including mix design variables such as water-to-cement ratio, cement content, supplementary cementitious materials, and aggregate type, in addition to handling, placement, and curing methods. Significant amounts of shrinkage can cause severe cracking, especially in the case of a bridge deck, later leading to poor durability performance. Drying shrinkage, due to moisture evaporation, usually comprises the most significant amount of shrinkage endured by the concrete. However, with high-performance concrete, when the water-to-cement ratio is low and water is therefore scarce, autogenous shrinkage, due to self-desiccation of the concrete, can be of the same magnitude as drying shrinkage and should also be evaluated.

Furthermore, several studies have shown an internal curing effect within the internal matrix of the concrete due to the increased amount of absorption of the lightweight aggregate. While conventional curing methods focus on preventing moisture from evaporating from the concrete surface, water stored in the internal structure of the lightweight aggregate can provide an additional supply of water to the cement paste once the cement hydration reaction has begun. Especially with high-performance concrete, that usually contains a low water-to-cement ratio, the water within the paste matrix is scarce. If not cured under ideal curing conditions, where the already scarce water is allowed to further evaporate, then enough water may not be present to allow the cement to fully complete the hydration process, thus not achieving the full-strength capacity of the material. However, if saturated lightweight aggregate is present throughout the concrete matrix, then the water within the aggregate pores is pulled out by capillary forces to provide a resource of stored water to aide in the curing process. This effect can be measured by the shrinkage, or lack thereof, of the concrete.

Drying Shrinkage

Drying shrinkage of both the optimized bridge deck and prestressed concrete mixtures was evaluated according to AASHTO T160, with the exception of using vibrating wire strain gages to measure the length change rather than the standard length comparator method. Three, 3-in. x 4-in. x 16-in. drying shrinkage (DS) beams were made for each concrete mixture and labeled with the designation DS1, DS2, and DS3. Each beam contained a 6-in. long 4200 vibrating wire strain gage (VWSG) obtained from Geokon. Using VWSGs, strains are measured from a length of steel wire tensioned between the two ends of the gage that are firmly in contact with the concrete. Deformations in the concrete cause the gage ends to move relative to one another, altering the tension in the wire. This change in tension is measured as a change in the resonant frequency of vibration of the wire. Electromagnetic coils are then used to obtain the readout of the gage frequency (Geokon, 2007). A hand-held portable reader from Geokon, model GK-404, was then used to take readings. To cast the beams, VWSGs were positioned on bar chairs and concrete was placed and consolidated around the gages in two layers. Care was taken to place the gages longitudinally in the beams and to not damage them during concrete placement. A picture of a VWSG and the beams during casting is shown in Figure 5.9 and Figure 5.10.

Figure 5.9. Vibrating wire strain gages before concrete placement.

Figure 5.10. Shrinkage beams during casting with VWSGs.

Initial gage readings were taken as soon as the concrete reached initial set. The VWSGs also contain a thermistor, so temperature variation was recorded and compensated for in the overall length change. After initial readings were taken, the DS beams were covered with plastic and cured in a moist room for 18-24 hours. Gage readings were taken periodically throughout the first day. At the age of 18-24 hours, the beams were then demolded and placed in a lime-water storage tank for 28 days. A picture of the beams stored in the lime water storage tank is shown in Figure 5.11. After 28 days, the beams were removed from the lime-water storage tank and cured in 50% relative humidity and 73°F. Readings were taken at 7, 14, 21, 28, 32, 35, 42, 49, 56, 84 and 140 days. It should be noted that this test measured total shrinkage experienced by the concrete. This means that both autogenous and drying shrinkage were measured.

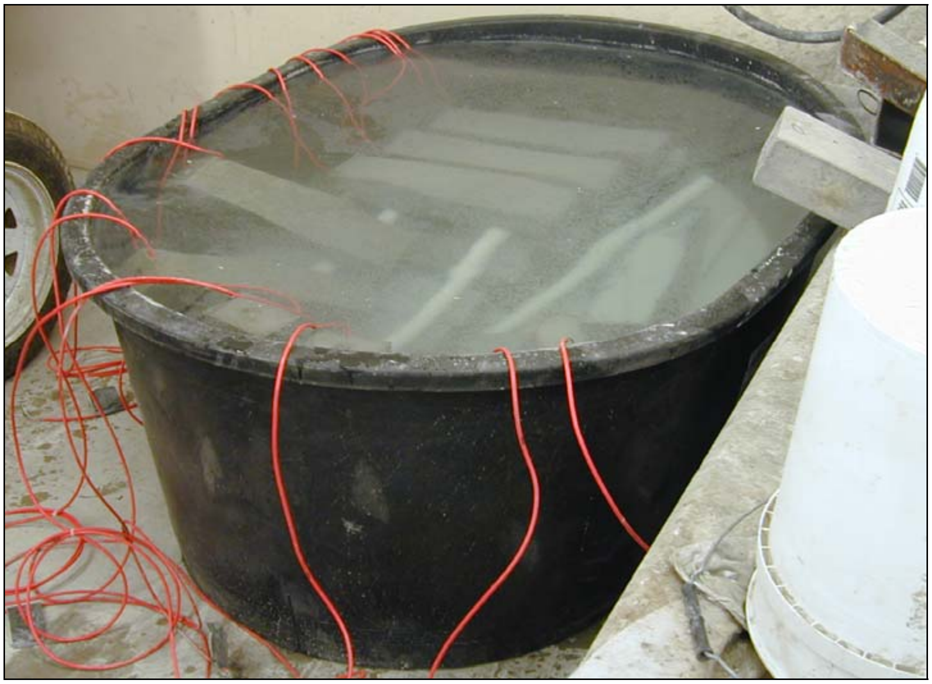

Figure 5.11. Lime water storage tank for drying shrinkage beams.

Autogenous Shrinkage

Autogenous shrinkage is defined as the change in volume produced by the continued hydration of cement, exclusive of the effects of applied load and change in either thermal condition or moisture content (Zhang, Li and Paramasivam 86). Autogenous shrinkage is caused by self-desiccation of the cement paste. Self-desiccation occurs if no excess water is supplied to the cement paste, and the process of chemical shrinkage makes intrinsic voids increase after the framework of hydrate is formed by setting (Tazawa and Miyazawa 15). Autogenous shrinkage can only be measured if no water is allowed to evaporate from the concrete, otherwise drying shrinkage occurs. Alternatively, no extra water should be supplied to concrete, so autogenous shrinkage can only be measured from specimens with no external moisture transfer. With conventional concrete, autogenous shrinkage is small compared to drying shrinkage. However, with high-performance concrete, where water-to-cement ratios are typically low and water is scarce, autogenous shrinkage can be as much as drying shrinkage, leading to potential cracking and durability concerns.

For this project, autogenous shrinkage was measured for both the optimized bridge deck and optimized prestressed concrete mixtures. As with the DS beams, three, 3-in. x 4-in. x 16-in. beams were made for each mixture, as designated AS1, AS2, and AS3. Each beam contained a VWSG placed longitudinally and cast as described above. Initial gage readings were taken as soon as the concrete reached initial set. After initial set, the beams were covered with plastic and cured in a moist room for 18-24 hours, then demolded. Gage readings were taken periodically for the first day and after being demolded. In order to measure the autogenous shrinkage and sustain a constant amount of moisture in contact with the concrete, these beams were wrapped 12-15 times with 0.003-in. thick polyolefin plastic wrap. A picture of the wrapped and sealed AS beams is shown in Figure 5.12. Each beam, with gage and plastic wrap, was weighed and placed in the moist room. The weight was evaluated again at the end of testing to determine if the beams were successfully sealed from moisture transfer. Readings were taken every day for the first week, then at 7, 14, 21, 28, 31, 35, 42, 49, 56, 84 and 140 days. A picture of the VWSGs being read in the shrinkage beams is shown in Figure 5.13.

Figure 5.12. Wrapped and sealed autogenous shrinkage beams in moist room.

Figure 5.13. VWSGs being read in shrinkage beams.

After the initial sets of autogenous and drying shrinkage beams were made and readings evaluated for approximately three months, it was decided to create three additional concrete mixtures to evaluate the autogenous and drying shrinkage, in order to compare results from the optimized bridge deck and optimized prestressed concrete mixtures originally created. The first additional concrete mixture consisted of redoing the optimized prestressed Stalite concrete mixture since confusing results were obtained during the original test. The second concrete mixture was the optimized bridge deck mixture containing Marquette aggregate with a different source of sand, obtained from Nebraska, to evaluate the effect of a different source of sand on the shrinkage. Finally, the third concrete mixture was the optimized bridge deck concrete mixture with the coarse lightweight aggregate replaced by crushed gravel, obtained from KDOT, to evaluate the effect of normal-weight aggregate on shrinkage.

Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) was determined for both the optimized bridge deck and optimized prestressed concrete mixtures for each type of lightweight aggregate. To determine the CTE, one of the three DS beams from each concrete mixture, created for the original shrinkage test, was used. All of these beams contained a VWSG, as discussed in the previous *Shrinkage* section, used to obtain a strain and temperature reading. The beams were taken after the shrinkage reading at an age of 140 days. The DS beams had been cured in lime-water until 28 days, then stored in air at 50% relative humidity and 73°F. The DS beams selected for the CTE experiment were then submerged in lime-water for 2 weeks before collecting CTE data to ensure there would be little effect of length deformation due to swelling.

The CTE test consisted of placing the beams in an environmental chamber, while keeping them submerged under water to prevent length deformation due to shrinkage. A temperature range of 33°F to 130°F was used in the environmental chamber. Test procedure consisted of taking the beams from room temperature, approximately 73°F, to the cold extreme of 33°F, and recording the corresponding temperature and strain before and after the change in temperature. Strain and temperature readings were monitored and recorded every hour throughout the day until the temperature within the beams had stabilized. Final readings were recorded and the beams were allowed to return to room temperature overnight. The following day, the same procedure was followed except the temperature range went from room temperature, approximately 73°F, to the hot extreme of 130°F. This procedure was repeated six times, allowing data to be acquired for three cold and three hot temperature swings.

Interestingly, from the raw data obtained it appears that the concrete expands as the temperature decreases and shrinks as the temperature increases. However, this phenomenon is due to the CTE of the VWSG being larger than the CTE of the concrete. Therefore, the raw data obtained, which is the length deformation sensed by the VWSG, must be converted to the actual strain undergone by the concrete by accounting for the difference in expansion of the VWSG and the concrete. The CTE was then determined using the actual strain undergone by the concrete and the equation

$$CTE = \left(\frac{\Delta L}{\Delta T} \right)$$

where
$$\begin{aligned} CTE &= \text{Coefficient of Thermal Expansion} \\ \Delta L &= \text{Change in Strain experienced by concrete} \\ \Delta T &= \text{Change in Temperature} \end{aligned}$$

CTE values obtained for each concrete mixture were then averaged. The average value for each concrete mixture was then used to analyze the shrinkage data.

CHAPTER 6 - Lightweight Concrete Properties and Experimental Results

This chapter gives experimental results of lightweight concrete properties tested during this project. Both optimized bridge deck and optimized prestressed lightweight concrete mixtures, discussed in Chapter 4, were evaluated for several concrete properties. Overall, concrete properties evaluated showed acceptable results within KDOT specification limits.

Moisture Content

Moisture content of the lightweight aggregate was determined at time periods of 1, 3, 7, 10, 14, 21, 28, and 60 days in both wet and SSD conditions. Moisture content results for each type of lightweight aggregate are shown in Table 6.1. Figure 6.1 shows how wet and SSD moisture content values change over time. It should be noted that values for day 14 seemed to be inconsistent with the trend for the rest of the wet and SSD moisture content values. The day 14 moisture content test was conducted by a different person, thus a slightly different procedure resulted in skewed moisture content values.

Table 6.1. Moisture content results for wet and SSD lightweight aggregate.

	Percent Moisture Content					
Day	**Marquette Wet**	**Marquette SSD**	**New Market Wet**	**New Market SSD**	**Stalite Wet**	**Stalite SSD**
1	23.5	17.5	16.2	10.3	7.3	4.0
3	27.1	20.5	18.5	12.4	7.8	3.9
7	30.0	22.9	20.4	14.4	8.7	5.7
10	32.9	25.8	22.7	16.6	10.0	5.9
14	30.5	25.5	21.5	16.3	8.3	4.8
21	34.0	27.8	26.7	20.7	9.5	5.2
28	35.1	28.4	25.5	19.8	9.5	5.9
60	37.6	30.9	28.9	23.7	11.6	6.8

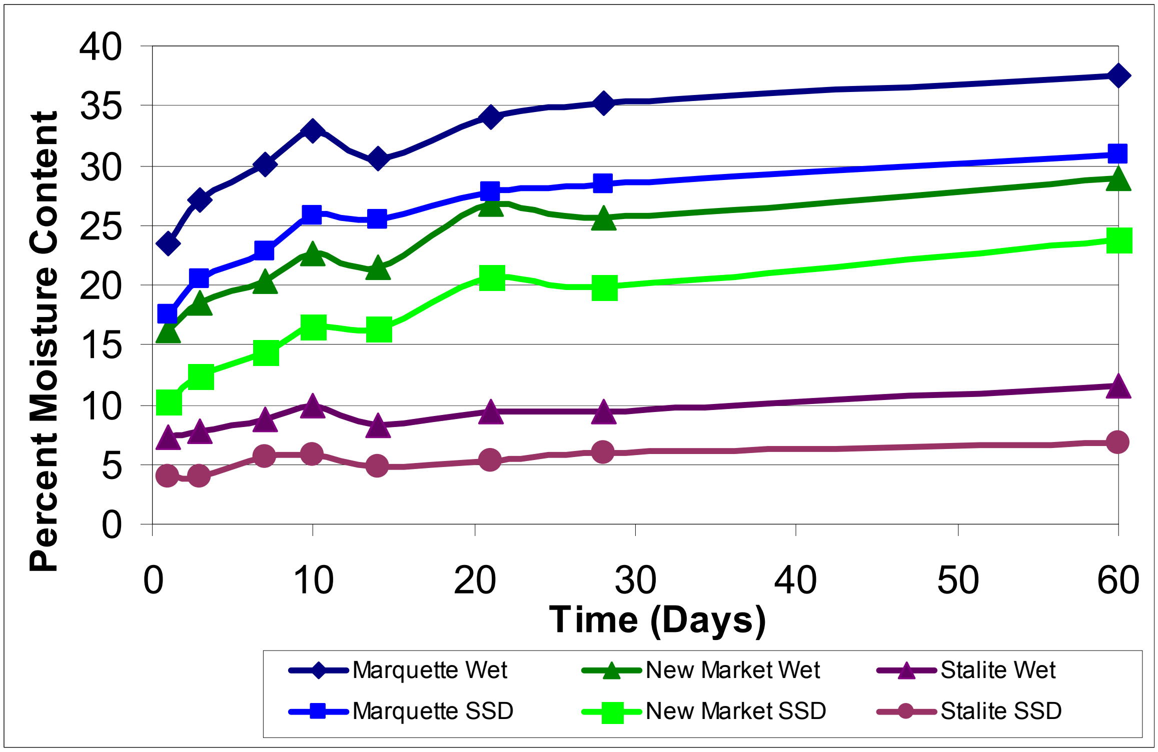

Figure 6.1. Results of wet and SSD moisture content over time.

In addition, what is likely more important than how moisture content values change over time, is how the difference between wet and SSD moisture contents change over time. The difference between the wet and SSD moisture content is the free water available in the concrete mixture to alter the water-to-cement ratio. This difference, over time, is shown in Figure 6.2. As can be seen in the graph, the difference between the wet and SSD moisture content is relatively constant, mostly varying within 1%. This should be expected since the free surface water should be similar for a given type and gradation of lightweight aggregate. The important issue is that the moisture content and absorption of the aggregate can be accurately accounted for in the concrete mix design and that the test procedure and results were consistent.

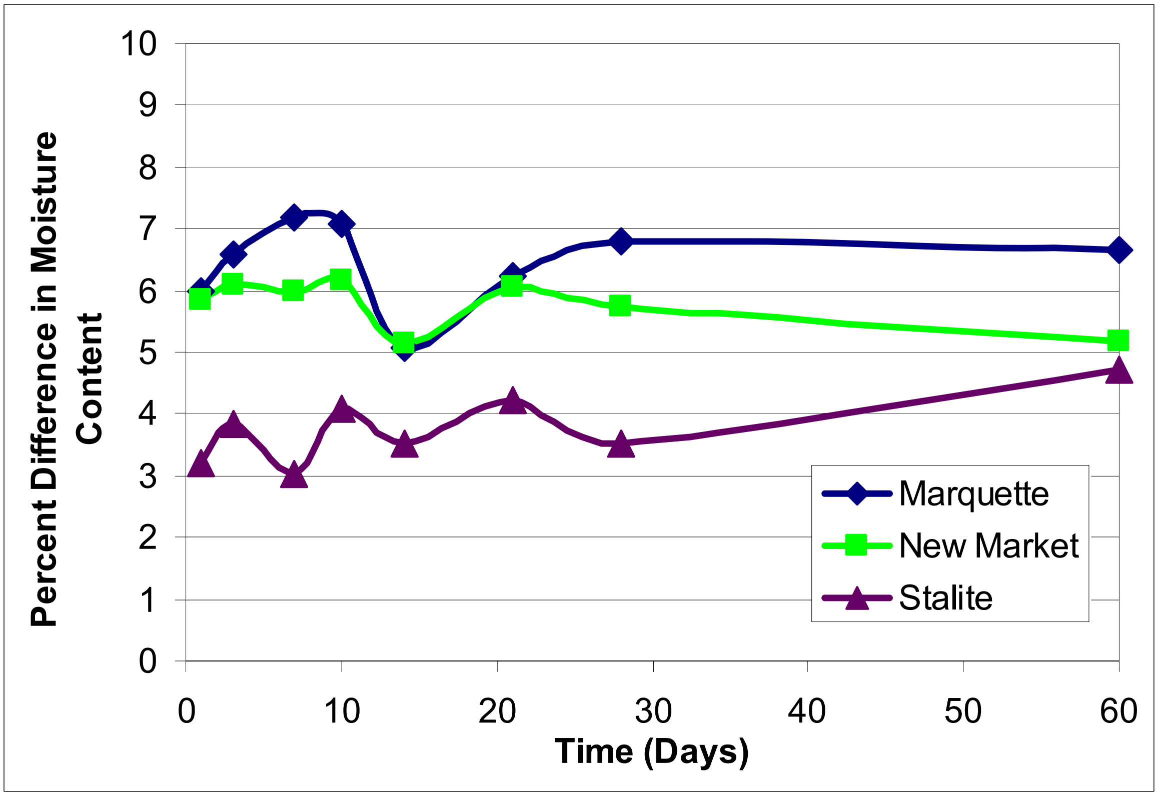

Figure 6.2. Difference in moisture content between wet and SSD aggregate.

Concrete mixes were tested with these new moisture content and absorption values. Experimental results revealed that slightly more water was needed in the Marquette and New Market concrete mixtures to produce slump and compressive strength results consistent with previous mixes. The amount of water added to the batch for each type of aggregate was back-calculated to determine the amount of water needed. This difference was likely due to the wet moisture content of the Marquette and New Market lightweight aggregates being slightly off. For both aggregates, the difference was determined to be approximately 1.5%. For this reason, the rest of the concrete mixtures made without using aggregate in the SSD condition, were designed with wet moisture contents that were 1.5% less than the measured wet moisture content values shown in Table 6.1. The wet and SSD moisture content values actually used in the successful concrete mix designs are given in Table 6.2. Several optimized bridge deck and prestressed concrete mixtures designed using these values successfully resulted in consistent slump and compressive strength values.

Table 6.2. Wet and SSD moisture content values used in concrete mixture designs.

Day	Marquette Wet	Marquette SSD	New Market Wet	New Market SSD	Stalite Wet	Stalite SSD
	Percent Moisture Content					
1	22.0	17.5	14.7	10.3	7.3	4.0
3	25.6	20.5	17.0	12.4	7.8	3.9
7	28.5	22.9	18.9	14.4	8.7	5.7
10	31.4	25.8	21.2	16.6	10.0	5.9
14	29.0	25.5	20.0	16.3	8.3	4.8
21	32.5	27.8	25.2	20.7	9.5	5.2
28	33.6	28.4	24.0	19.8	9.5	5.9
60	36.1	30.9	27.4	23.7	11.6	6.8

Compressive Strength

Compressive strength was determined at time periods of 1, 3, 7, 14, 21, and 28 days for the optimized bridge deck mixtures and 16-hours, 3, 7, 14, 21, and 28 days for the optimized prestressed mixtures, for each type of lightweight aggregate. Procedure details are discussed in Chapter 5. All compressive strength, tensile strength, and modulus of elasticity specimens were made from the same batch of concrete for each type of lightweight aggregate. Concrete property results for the optimized bridge deck concrete mixtures are given in Table 6.3. Corresponding compressive strength results for the optimized bridge deck concrete mixtures are shown in Table 6.4. A graph of the strength gain over time is shown in Figure 6.3.

Table 6.3. Optimized bridge deck fresh concrete properties.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)
Marquette	5	135.1	5
New Market	4.5	132.9	4
Stalite	6.5	132.4	8

Table 6.4. Optimized bridge deck concrete compressive strength.

Day	Compressive Strength (psi)		
	Marquette	New Market	Stalite
1	1160	1360	1280
3	2330	2430	2110
7	2740	3330	2900
14	4230	4070	3470
21	4510	4570	4190
28	5050	4910	4160

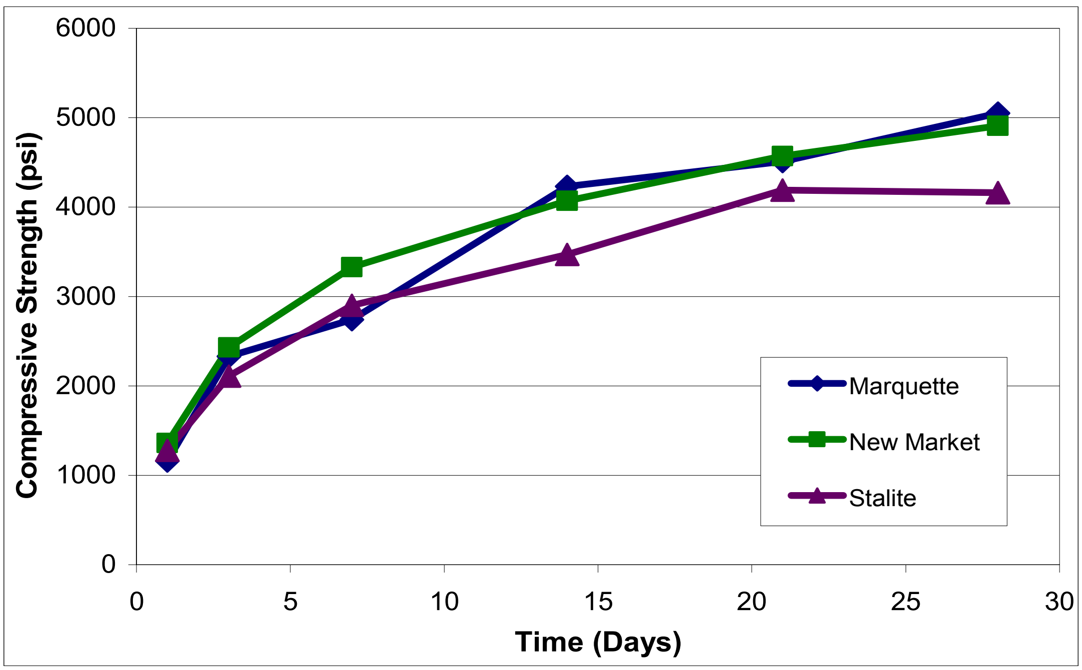

Figure 6.3. Bridge deck concrete compressive strength gain over time for all three aggregate types.

One of the goals for the bridge deck concrete mixtures was a 28-day compressive strength of 4000 psi, which corresponds to a laboratory compressive strength of 5200 psi. As shown, none of the optimized bridge deck concrete mixtures meet this requirement for the data shown in Table 6.4. However, this data was chosen to be reported because the same batch of concrete was used for compressive strength, tensile strength, and modulus of elasticity. These compressive strength cylinders were first used to determine the modulus of elasticity. While determining the modulus of elasticity should not affect the compressive strength, it is believed that some microcracking occurred and therefore reduced the compressive strength. Several other optimized bridge deck concrete mixtures, made with the same proportions, resulted in compressive strengths over 5200 psi. An example of optimized bridge deck mixtures that achieved acceptable compressive strength is given in Table 6.5. Nevertheless, compressive strength results reported in Table 6.4 adequately demonstrate strength gain over time of the optimized bridge deck concrete mixtures for each type of lightweight aggregate. A complete list of all concrete mixtures and results is given in Appendix A - Concrete Mixture Summary.

Table 6.5. Optimized bridge deck concrete with acceptable compressive strength.

	Compressive Strength (psi)		
Day	**Marquette**	**New Market**	**Stalite**
7	2550	3820	3580
28	5410	6210	5220

Since all compressive strength, tensile strength, and modulus of elasticity specimens were made from the same batch, concrete property results for the optimized prestressed concrete mixtures are given in Table 6.6. Corresponding compressive strength results for the optimized prestressed concrete mixtures are shown in

Table 6.7. All optimized prestressed concrete mixtures met the design goal of a 16-hour 5000 psi compressive strength. A graph of the strength gain over time is shown in Figure 6.4.

Table 6.6. Optimized prestressed fresh concrete properties.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)
Marquette	4.0	122.4	3
New Market	6.0	115.0	9
Stalite	7.0	116.6	9

Table 6.7. Optimized prestressed concrete compressive strength.

Day	Compressive Strength (psi)		
	Marquette	New Market	Stalite
16-hour	5360	5370	5160
3	6540	6660	6780
7	7330	6820	7290
14	7360	7200	7570
21	7740	6920	8060
28	7550	7200	8230

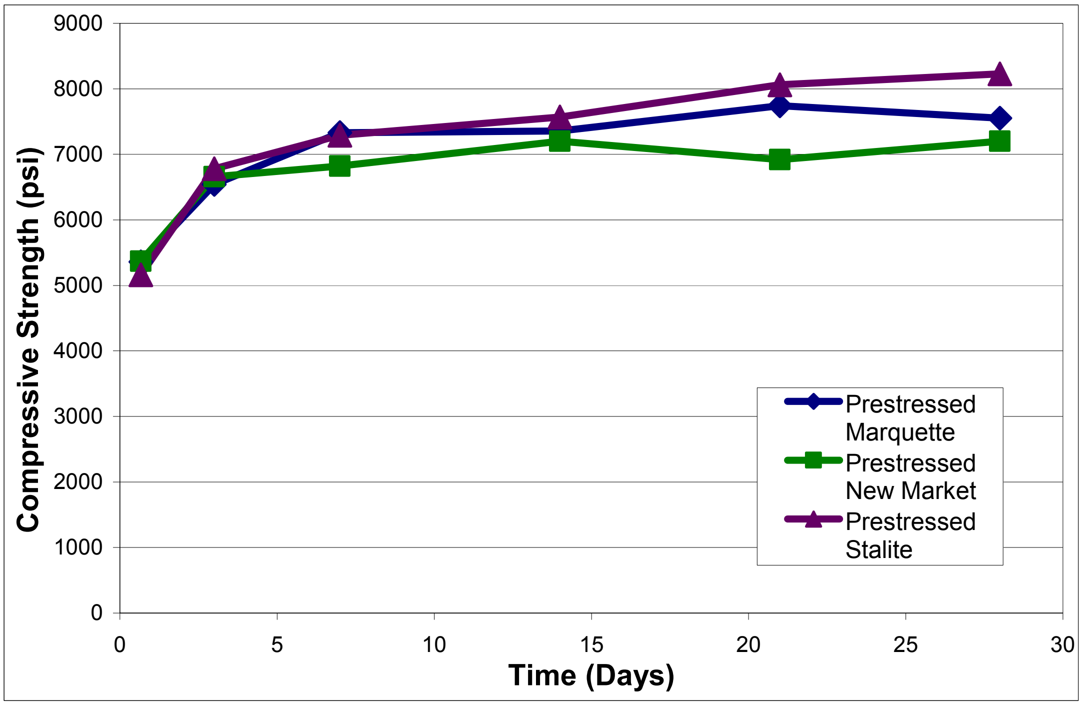

Figure 6.4. Prestressed concrete compressive strength gain over time for all three aggregate types.

Tensile Strength

Tensile strength was determined at time periods of 1, 7, 14, 21, and 28 days for the optimized bridge deck mixtures and 16-hours, 3, 7, 14, 21, and 28 days for the optimized prestressed mixtures, for each type of lightweight aggregate. The splitting tensile test method was used. Procedure details are discussed in Chapter 5. All compressive strength, tensile strength, and modulus of elasticity specimens were made from the same batch of concrete for each type of lightweight aggregate. Fresh concrete property results for the optimized bridge deck concrete mixtures are given in Table 6.3.

Tensile strength results for the optimized bridge deck concrete mixtures are shown in Table 6.8. Due to batching difficulties, batch size was reduced and the day-1, Stalite tensile strength specimens were not made or tested.

Table 6.8. Optimized bridge deck concrete tensile strength results.

	Tensile Strength (psi)		
Day	**Marquette**	**New Market**	**Stalite**
1	150	140	-
7	270	330	260
14	320	320	310
21	390	390	300
28	380	330	330

Tensile strength results for the optimized prestressed concrete mixtures are shown in Table 6.9. Fresh concrete property results for the optimized prestressed mixtures are given in Table 6.6.

Table 6.9. Optimized prestressed concrete tensile strength results.

	Tensile Strength (psi)		
Day	**Marquette**	**New Market**	**Stalite**
16-hour	400	360	310
3	340	390	440
7	450	430	480
14	410	390	470
21	500	470	480
28	430	460	420

Modulus of Elasticity

The modulus of elasticity was determined at time periods of 1, 3, 7, 14, 21, and 28 days for the optimized bridge deck mixtures and 16-hours, 3, 7, 14, 21, and 28 days for the optimized prestressed mixtures, for each type of lightweight aggregate. Procedure details and test setup are discussed in Chapter 5. Concrete property results for the optimized bridge deck concrete mixtures are given in Table 6.3. Modulus of elasticity results for both E_{chord} and E_{graph} for the optimized bridge deck concrete mixtures are shown in Table 6.10.

Table 6.10. Optimized bridge deck concrete modulus of elasticity results.

Day	Modulus of Elasticity (ksi) Marquette E_{chord}	Marquette E_{graph}	New Market E_{chord}	New Market E_{graph}	Stalite E_{chord}	Stalite E_{graph}
1	1130	1120	1400	1390	1380	1360
3	2040	2060	2790	2730	2670	2660
7	2240	2080	2190	2120	2960	2830
14	2650	2570	2580	2530	2970	2930
21	2660	2640	2820	2780	2860	2870
28	2790	2770	2570	2550	2580	2550

As shown in the table, most of the E_{chord} and E_{graph} values are very similar. The 28-day results are approximately 2800 ksi for the Marquette aggregate, and 2600 for the New Market and Stalite aggregates. The standard value for the 28-day modulus of elasticity of normal-weight concrete is about 3600 ksi. The 28-day modulus of elasticity values obtained for the optimized bridge deck concrete is lower than the standard, normal-weight concrete modulus. The lower modulus of elasticity is expected for lightweight concrete since the lightweight aggregate itself has a lower modulus of elasticity. The modulus of elasticity of concrete is made up of the different moduli of elasticity of each of the different material constituents that make up the concrete composite. Since the modulus of the lightweight aggregate is lower than that of normal-weight aggregate, the resulting modulus of the concrete will also be lower. A lower modulus of elasticity can be beneficial in bridge deck applications. A lower modulus of elasticity means the material is more flexible and can sustain more load with less cracking, leading to improved durability.

On a microscopic level, the lower cracking phenomena can be explained by the fact that the modulus of elasticity of the lightweight aggregate is more closely matched to the modulus of elasticity of the paste than is normal-weight aggregate (Bremner and Holm, 1986). The result is that when a load is applied to the concrete, the composite material is able to deform more uniformly and less microcracking occurs within the contact zone between the aggregate and paste. Again, less microcracking results in improved concrete durability. In addition, finely ground lightweight aggregate, namely expanded clay, shale, and slate, has shown to be pozzolanic, meaning that it will form a bond using the byproducts of cement hydration (Holm and Bremner, 1984). This property, in addition to the lightweight aggregate having a similar stiffness to the surrounding matrix, allows for better bond within the contact zone. Bremner et al. (1984) studied aggregate-matrix interaction within the contact zone and concluded that the lightweight concrete studied developed sufficient bond between the aggregate and mortar matrix compared to normal-weight concrete. For this project, pictures were taken of polished cylinder cross-sections for each type of lightweight aggregate using a scanning-electron microscope. Pictures of the concrete cylinder cross-sections are shown in Figure 6.5, Figure 6.6, and Figure 6.7, for the Marquette, New Market, and Stalite aggregates, respectively. The red circle on each figure represents the area of the cross-section where the scanning-electron microscope pictures were focused.

Figure 6.5. Polished cylinder cross-section of Marquette aggregate concrete.

Figure 6.6. Polished cylinder cross-section of New Market aggregate concrete.

Figure 6.7. Polished cylinder cross-section of Stalite aggregate concrete.

Scanning-electron microscope pictures are shown with scales of 1mm for all three types of lightweight aggregate and 500µm for the Marquette aggregate and 400µm for the New Market and Stalite aggregates. Pictures of the Marquette aggregate lightweight concrete is shown in Figure 6.8 and Figure 6.9. Pictures of the New Market lightweight concrete is shown in Figure 6.10 and Figure 6.11. Finally, pictures of the Stalite lightweight concrete is shown in Figure 6.12 and Figure 6.13.

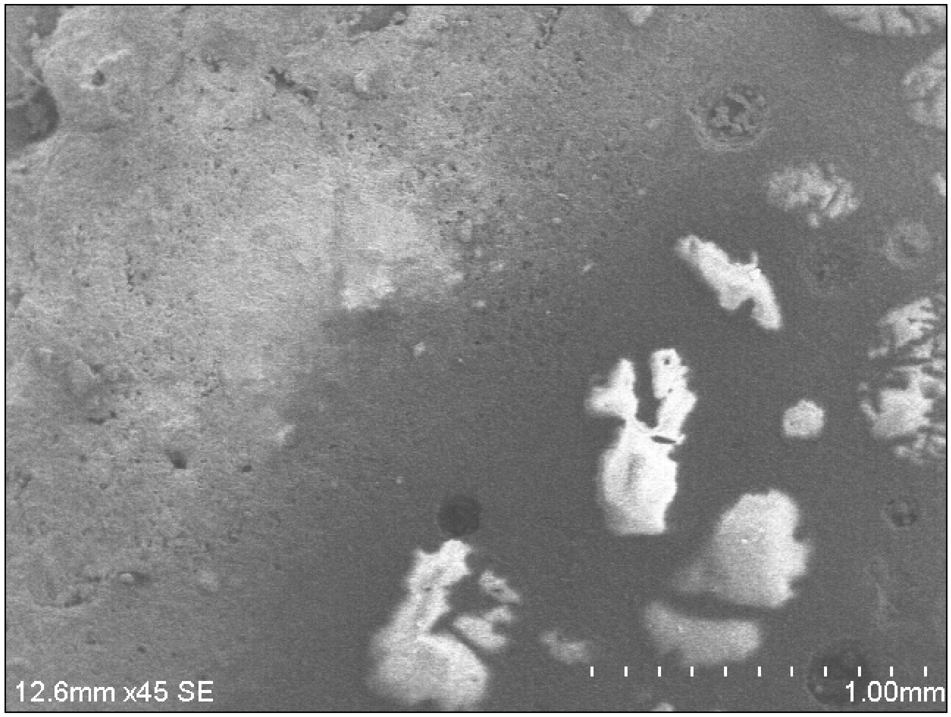

Figure 6.8. Contact zone of Marquette aggregate concrete with 1mm scale.

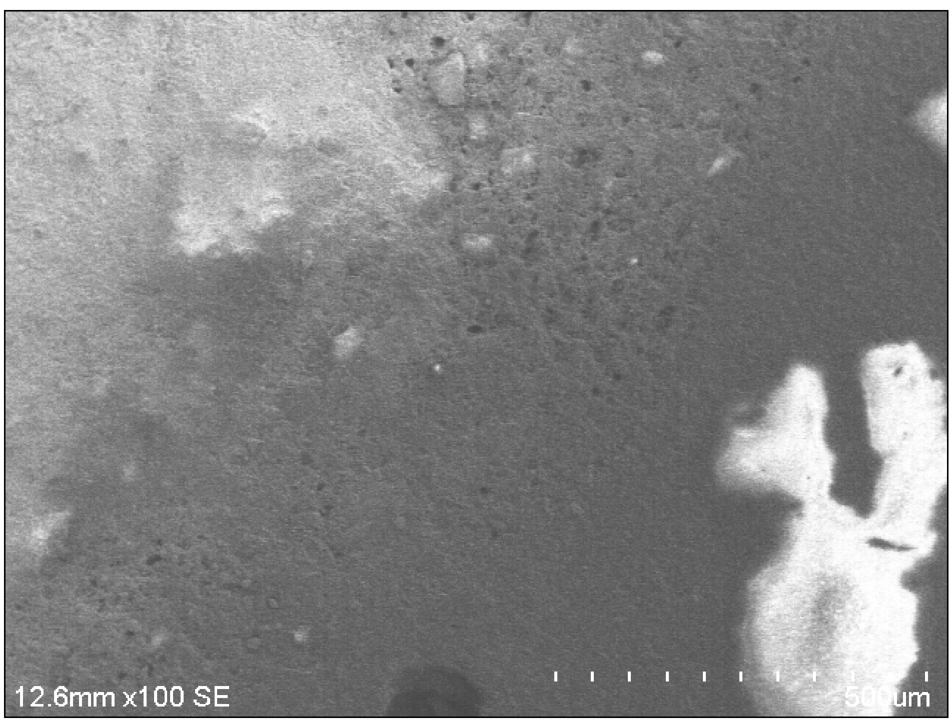

Figure 6.9. Marquette aggregate concrete with 500µm scale.

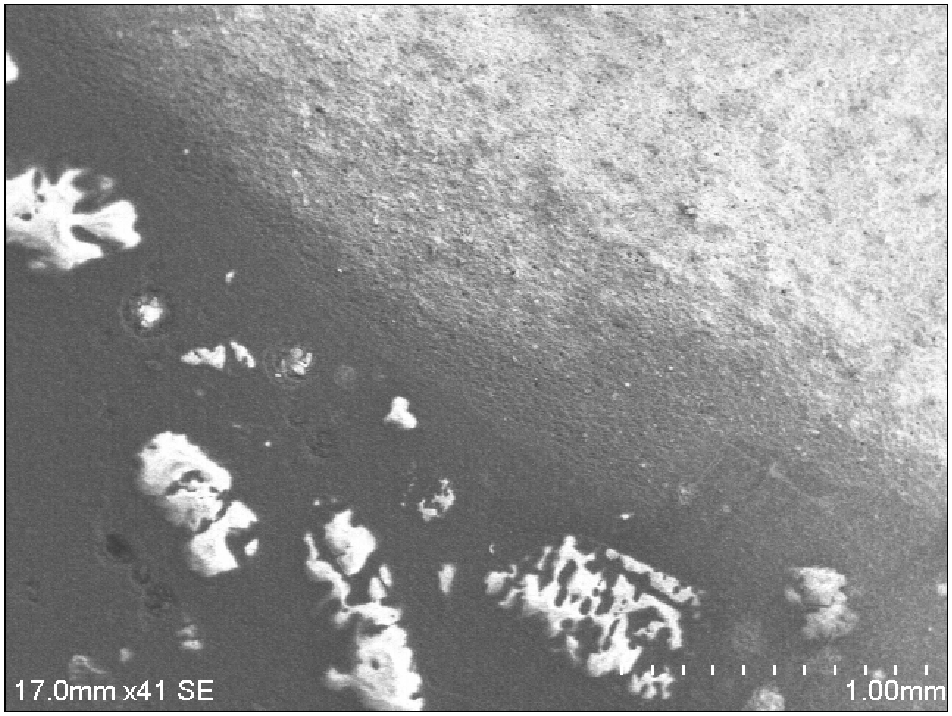

Figure 6.10. Contact zone of New Market aggregate concrete with 1mm scale.

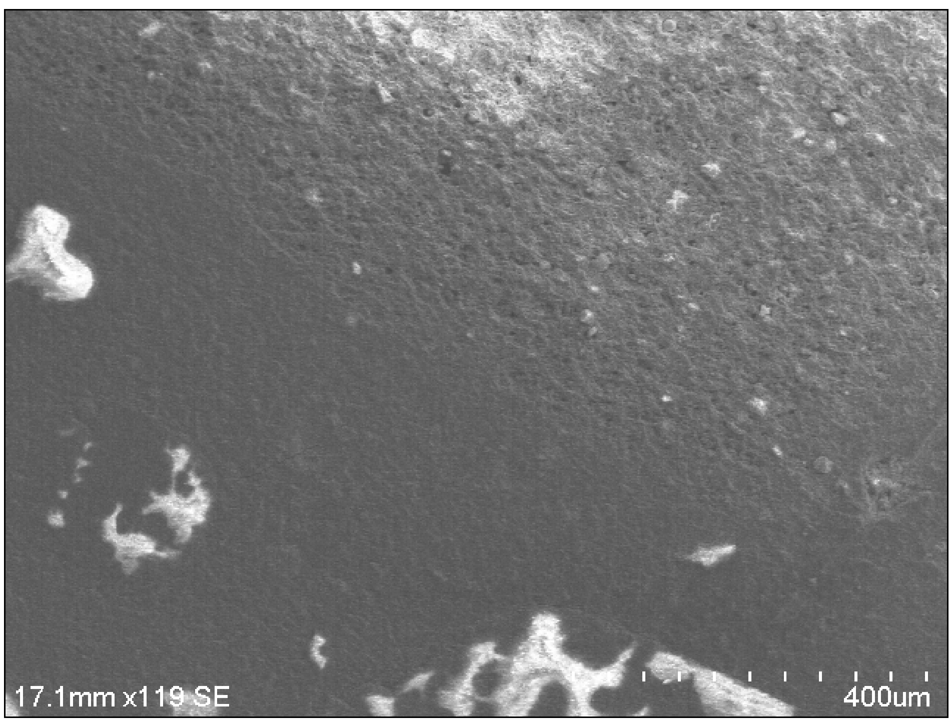

Figure 6.11. New Market aggregate concrete with 400µm scale.

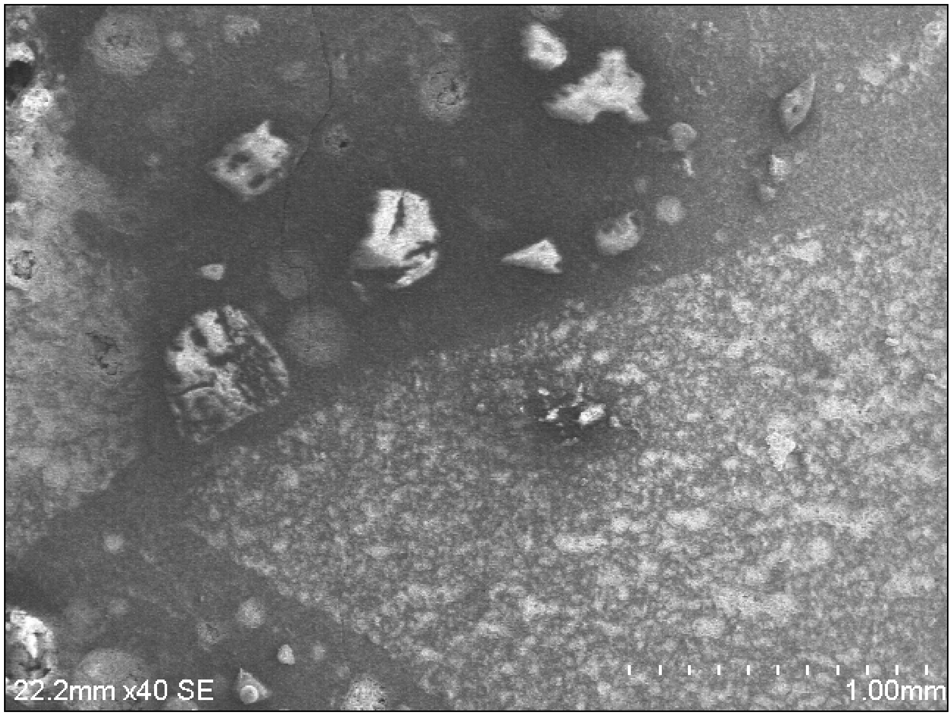

Figure 6.12. Contact zone of Stalite aggregate concrete with 1mm scale.

Figure 6.13. Stalite aggregate concrete with 400μm scale.

Each of the scanning-electron microscope pictures are focused on the contact zone between the lightweight aggregate and the surrounding paste matrix. The paste matrix can be distinguished in each figure by the presence of sand, appearing as white objects. The sand appears white due to the fact that the electrons in the scanning-electron microscope are more reactive to the silica that the sand is composed of and therefore are reflected back at a higher rate. With the Stalite aggregate, the division of the contact zone can be clearly seen. However, with the Marquette and New Market aggregates, the exact location of where the paste matrix ends and the aggregate begins is blurred. This indistinct division is an indication that the paste matrix has penetrated into the lightweight aggregate pores developing better bond within the contact zone. In addition, this blurred division could also indicate a slight pozzolanic reaction between the lightweight aggregate and the surrounding paste matrix, also producing better bond within the contact zone.

Fresh concrete property results for the optimized prestressed mixtures are given in Table 6.6. The optimized prestressed modulus of elasticity results for both E_{chord} and E_{graph} are shown in Table 6.11. The 28-day modulus of elasticity results for the optimized prestressed concrete is approximately 3600 ksi for all three lightweight aggregate types. Although lower modulus of elasticity values are expected from lightweight concrete, the higher modulus of elasticity values obtained are likely due to the high-strength nature of these prestressed concrete mixtures. Modulus of elasticity data for the Stalite prestressed concrete was not successfully obtained on day 3, and is therefore not reported.

Table 6.11. Optimized prestressed concrete modulus of elasticity results.

	Modulus of Elasticity (ksi)					
Day	**Marquette E_{chord}**	**Marquette E_{graph}**	**New Market E_{chord}**	**New Market E_{graph}**	**Stalite E_{chord}**	**Stalite E_{graph}**
16-hour	2730	2750	2610	2580	3150	3110
3	3130	3110	3140	3060	-	-
7	3380	3340	3720	3790	3370	3340
14	3480	3110	3420	3380	3040	3020
21	3110	3170	3490	3590	3160	3180
28	3630	3670	3690	3700	3410	3290

Freeze-Thaw Resistance

The resistance to freezing and thawing cycles test was conducted for the optimized bridge deck concrete mixtures. Procedure details and test setup are discussed in Chapter 5. The same batch of concrete was used to make test specimens for freeze-thaw resistance, permeability, and alkali-silica reactivity for each type of lightweight aggregate. Fresh concrete properties for these mixes are shown in Table 6.12. Durability factor and expansion are given in Table 6.13 for both sets of beams tested for each type of lightweight aggregate.

Table 6.12. Fresh concrete properties for optimized bridge deck concrete mixtures.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)
Marquette	6.0	121.9	5.50
New Market	7.5	124.3	4.75
Stalite	6.25	122.7	6.25

Table 6.13. Results for optimized bridge deck concrete at 300 cycles of freezing and thawing.

	Set 1	Set 2	Average
Marquette			
Durability Factor	98	95	96.5
Percent Expansion	0.044	0.025	0.035
New Market			
Durability Factor	100	100	100
Percent Expansion	0.013	0.012	0.013
Stalite			
Durability Factor	99	97	98
Percent Expansion	0.018	0.019	0.019

The KDOT 1102 specification for coarse aggregates for concrete gives acceptable durability factor values of 95 or more, and acceptable percent expansion values of 0.025% or less. Each set of all three types of lightweight aggregates have satisfactory durability factors. The New Market and Stalite aggregates performed very well, having average durability factors of 100 and 98, respectively. The Marquette aggregate also had an acceptable durability factor performance with an average value of 96.5, but set 2 was at the minimum acceptable value of 95. Furthermore, the New Market and Stalite aggregates also had satisfactory percent expansion values of 0.013% and 0.019%, respectively. However, the Marquette aggregate had an average percent expansion of 0.035%, which is above the KDOT specification limit of 0.025%. Since the Marquette aggregate had an acceptable durability factor, but a percent expansion that is over the KDOT limit, it is recommended that the Marquette aggregate be evaluated again for freeze-thaw resistance to determine if the aggregate is satisfactory.

Permeability

The *Electrical Indication of Concrete's Ability to Resist Chloride-Ion Penetration* test was conducted to determine the permeability for the optimized bridge deck concrete mixtures. Procedure details and test setup are discussed in Chapter 5. The same batch of concrete was used to make test specimens for freeze-thaw resistance, permeability, and alkali-silica reactivity for each type of lightweight aggregate. Fresh concrete properties for these mixes are shown in Table 6.12. Permeability results for all three types of lightweight aggregate are shown in Table 6.14.

Table 6.14. Results of electrical charge passed through specimens indicating chloride ion penetration.

	Permeability (coulombs)
Marquette	3660
New Market	3980
Stalite	3490

Correlation between the amount of electrical charge passed through the test specimen and resulting resistance to chloride-ion penetration is shown in Table 5.1. According to this table, all three lightweight aggregates are in the 2000 – 4000 coulomb range, indicating they have moderate permeability. These are acceptable permeability values.

Alkali-Silica Reactivity

Alkali-silica reactive aggregate has been known to cause expansion and poor durability concrete. For this reason, the *Wetting and Drying Test of Sand and Sand-Gravel Aggregate for Concrete* test was conducted according to the KDOT KTMR-23 specification. Procedure details and test setup are discussed in Chapter 5. Since it is unlikely that the prestressed concrete beams would be exposed to a significant amount of wetting and drying, only the optimized bridge deck concrete mixtures were tested for alkali-silica reactivity. The same batch of concrete was used to make test specimens for freeze-thaw resistance, permeability, and alkali-silica reactivity for each type of lightweight aggregate. Fresh concrete properties for these mixes are shown in Table 6.12. Results of the wetting and drying test are shown in Table 6.15. Results for the 60-day flexural modulus of rupture for the Stalite aggregate were not recorded and are therefore not included.

Table 6.15. Alkali-silica reactivity results for optimized bridge deck concrete mixtures.

	60 Days	180 Days	365 Days
Marquette			
Percent Length Change	0.009	0.010	0.019
Flexural Modulus of Rupture (psi)	690	-	630
New Market			
Percent Length Change	0.008	0.011	0.019
Flexural Modulus of Rupture (psi)	750	-	800
Stalite			
Percent Length Change	0.004	0.009	0.011
Flexural Modulus of Rupture (psi)	-	-	730

Requirements for acceptability of the aggregate are based on results from beams tested at 60 days and at 365 days. Each set of beams must have a flexural modulus of rupture not less than 550 psi. In addition, the percent length change of the beams exposed to the wetting and drying test cycles must not exceed 0.050% at 180 days, nor 0.070% at 365 days. Based on these acceptability requirements, all three lightweight aggregate types showed satisfactory alkali-silica reactivity results.

Shrinkage

The drying shrinkage and autogenous shrinkage was measured for both the optimized bridge deck and optimized prestressed concrete mixtures for all three types of lightweight aggregate. For each concrete mixture, three drying shrinkage (DS) beams and three autogenous shrinkage (AS) beams were made and monitored as described in Chapter 5. In addition to the six original concrete mixtures created for shrinkage evaluation, three more concrete mixtures were created and monitored starting approximately three months after the original mixtures were made. The three additional concrete mixtures consisted of redoing the optimized prestressed Stalite mixture that had previously been done in the original mixes, and two variations on the optimized bridge deck concrete with Marquette aggregate. One variation of this mixture was replacing the sand with sand of a similar gradation from a different source. The other variation was replacing the Marquette aggregate with normal-weight gravel obtained from KDOT that also had a similar gradation. Fresh concrete properties of the optimized bridge deck shrinkage mixtures are shown in Table 6.16. Autogenous and drying shrinkage results for the three original bridge deck mixtures are shown in Figure 6.14, Figure 6.15, and Figure 6.16, for the Marquette, New Market, and Stalite aggregates, respectively.

Table 6.16. Concrete properties for optimized bridge deck shrinkage mixtures.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)	Compressive Strength 28-Day (psi)
Marquette	8.0	120.8	5.25	5600
New Market	7.5	121.1	5.25	5730
Stalite	6.5	120.0	6.0	5240

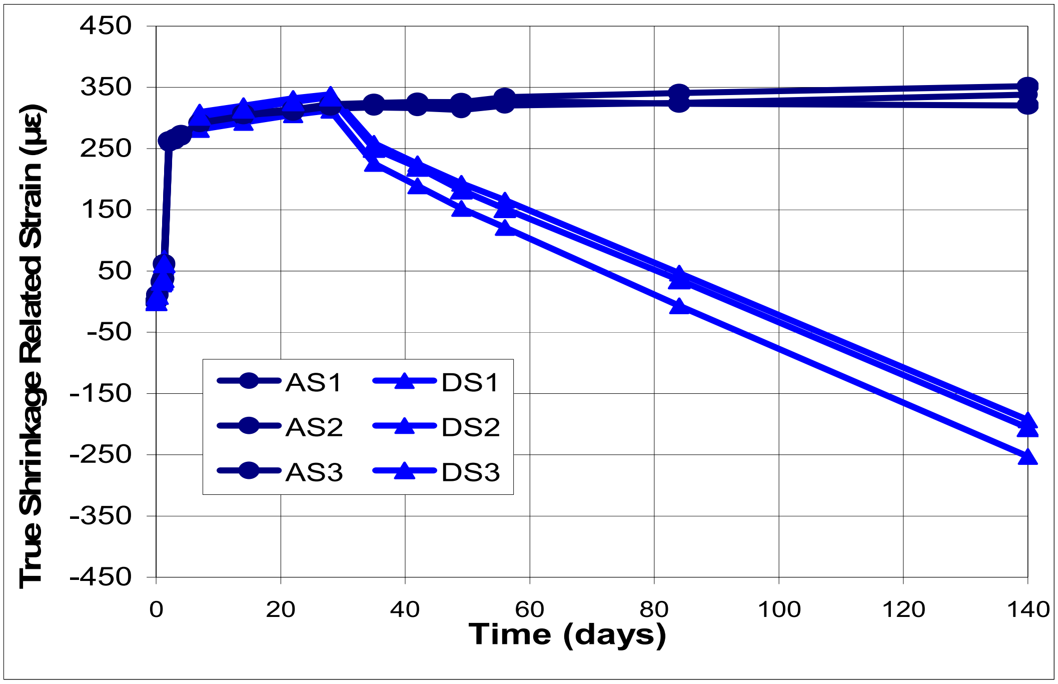

Figure 6.14. Marquette shrinkage results for optimized bridge deck concrete mixture.

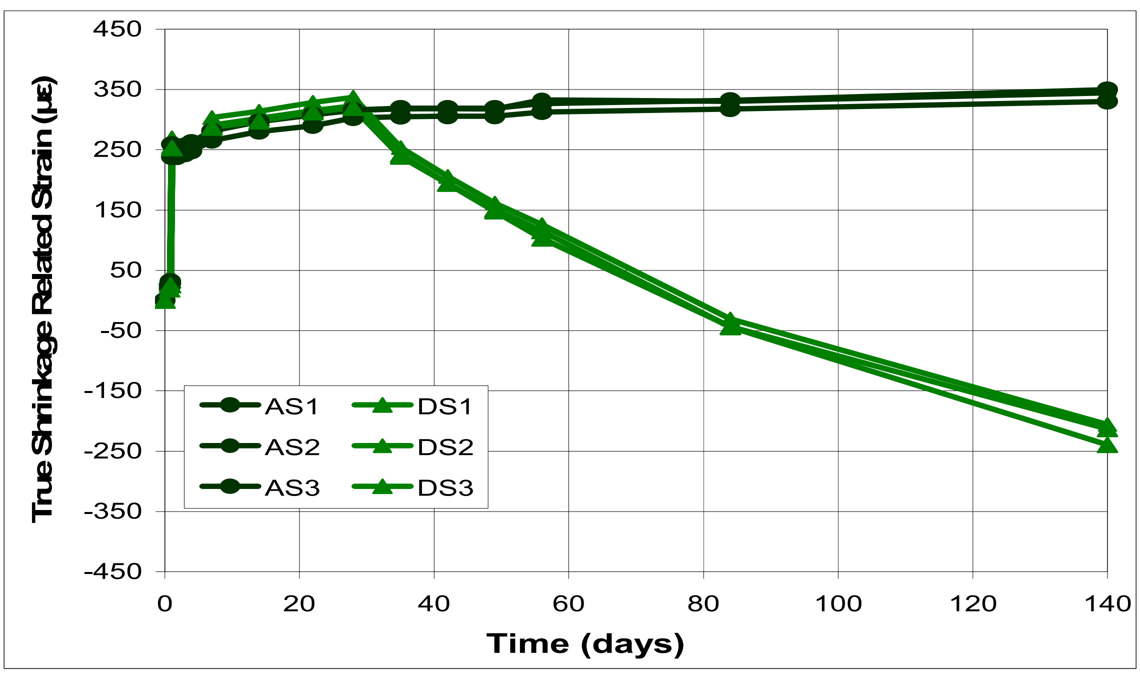

Figure 6.15. New Market shrinkage results for optimized bridge deck concrete mixture.

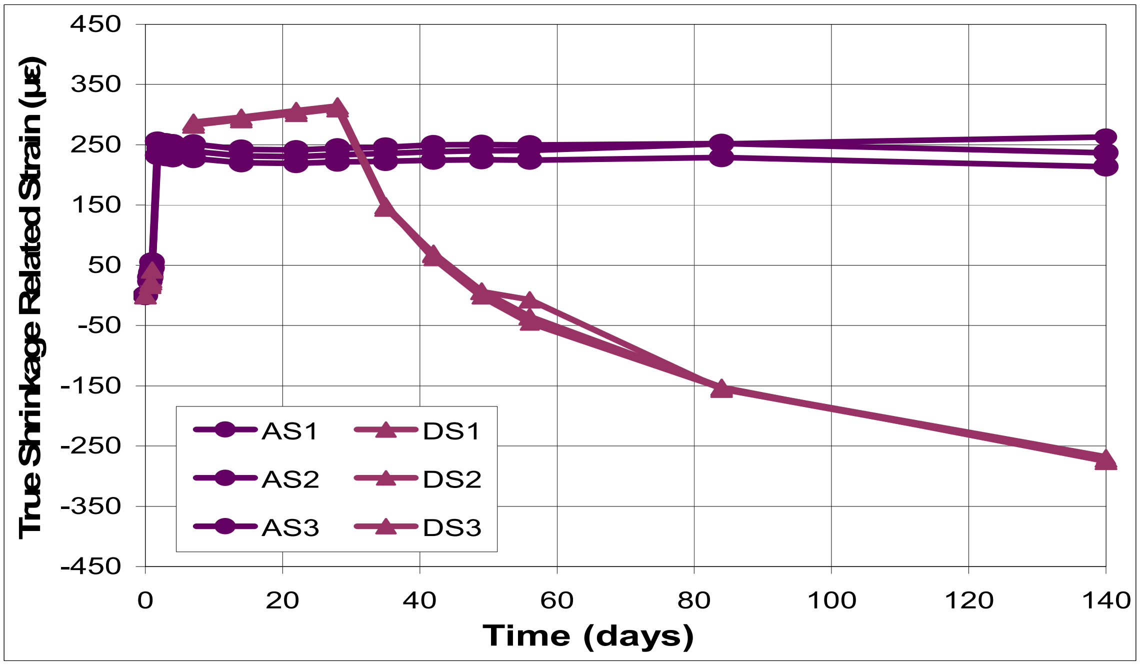

Figure 6.16. Stalite shrinkage results for optimized bridge deck concrete mixture.

All three lightweight aggregates behave similarly for the optimized bridge deck concrete mixtures. For the first 24 hours, while both the DS and AS specimens were in the molds, and treated the same, the concrete is shown to swell. The DS beams were then placed in the lime-water storage tank for 28 days, and they continued to swell slightly while in contact with free water. After 28 days, the DS beams were removed from the tank and stored at 73°F and 50% relative humidity. During this period, all of the DS beams began to experience drying shrinkage. At 84 days, all DS beams had experienced enough shrinkage to overcome the initial swelling and produce actual negative length change, or shrinkage values. The DS trend continues to decline for all three aggregates up to 140 days. The AS beams were wrapped and sealed with several layers of plastic after being demolded. After this point, all three lightweight aggregates, which had swelled in the first 24 hours, continued to expand, but at a much lower rate. This phenomenon of expansion while continued cement hydration and self-desiccation is occurring, is believed to be caused by the presence of absorbed moisture in the internal pore structure of the lightweight aggregate. It is important to note that both the Marquette and New Market aggregates experience approximately the same amount of expansion, 300 με, while the Stalite aggregate expansion is less, around 250 με. These results support the internal curing theory because the Marquette and New Market aggregates have higher absorption capacities than does the Stalite aggregate, providing more moisture internally to negate the effect of self-desiccation. A graph of the AS and DS averages for all three types of lightweight aggregate is shown in Figure 6.17.

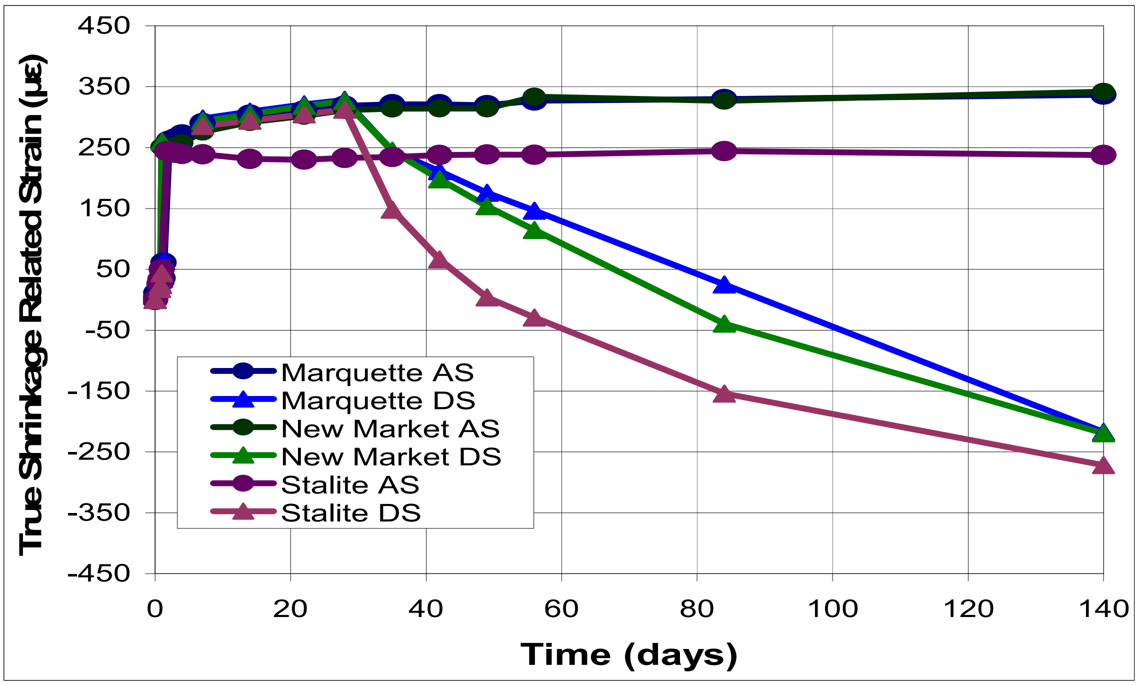

Figure 6.17. Average AS and DS shrinkage results for optimized bridge deck concrete for all three lightweight aggregates.

Fresh concrete properties for the optimized prestressed shrinkage mixtures is shown in Table 6.17. Autogenous and drying shrinkage results for the original optimized prestressed concrete mixtures are shown in Figure 6.18, Figure 6.19, and Figure 6.20, for the Marquette, New Market, and Stalite aggregates, respectively. Prestressed autogenous shrinkage and drying shrinkage beams are designated PAS and PDS, respectively.

Table 6.17. Concrete properties for optimized prestressed shrinkage mixtures.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)	Compressive Strength 16-Hour (psi)
Marquette	8.0	117.2	7.0	5480
New Market	8.0	116.2	7.0	5500
Stalite	6.0	117.0	9.0	5880

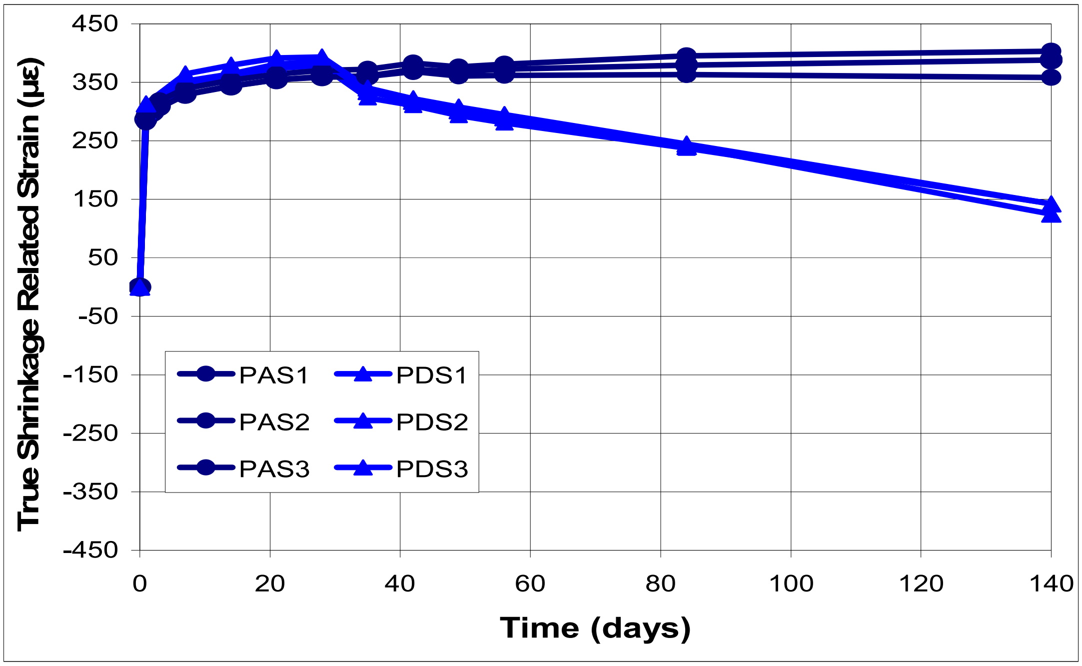

Figure 6.18. Marquette shrinkage results for optimized prestressed concrete mixture.

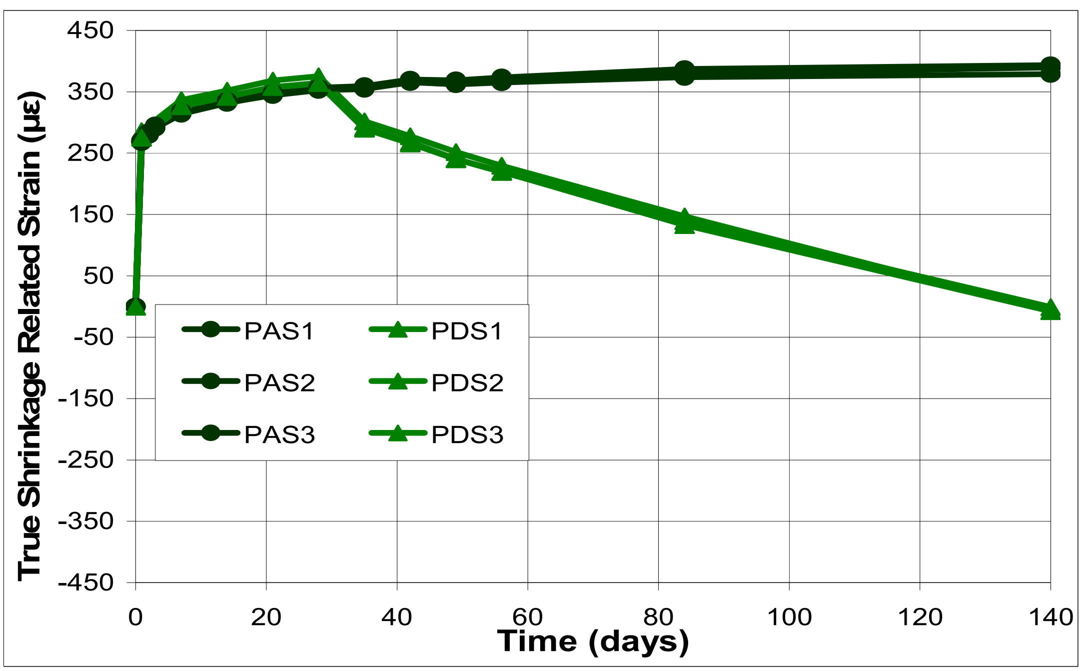

Figure 6.19. New Market shrinkage results for optimized prestressed concrete mixture.

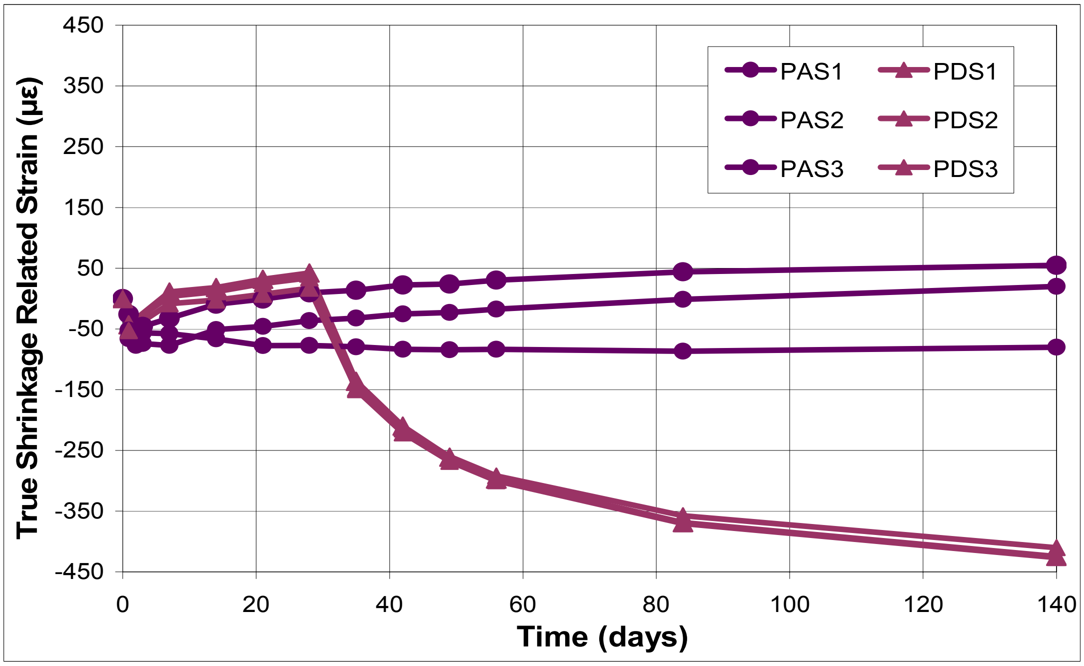

Figure 6.20. Stalite shrinkage results for optimized prestressed concrete mixture.

The optimized prestressed mixtures follow the same basic trend as the optimized bridge deck concrete mixtures, with the exception of the Stalite prestressed mixture. During the first 24 hours, both Marquette and New Market PDS and PAS beams expand, when all of the beams were in molds being cured in the moist room. After demolding, the wrapped and sealed PAS beams stay relatively the same length or continue to expand slightly. Both Marquette and New Market aggregates expanded to approximately 350 με. During the first two days, the Stalite aggregate experienced a small amount of shrinkage, about 60 με, then began to expand at similar rates to the Marquette and New Market aggregates. The PDS beams for each aggregate were placed in the lime-water storage tank for 28 days where they continued to expand in the presence of free water. After 28 days, the PDS beams were removed from the lime-water and drying shrinkage was observed. At the age of 140 days, the Marquette PDS beams had experienced significant drying shrinkage, but still had a positive overall length change, meaning that the total length deformation was still expansion. The New Market PDS beams had undergone enough drying shrinkage at 140 days that the overall length change was actual shrinkage. The prestressed Stalite PDS beams experienced the most overall shrinkage with shrinkage of almost 450 με at an age of 140 days. The averages for the three PAS and PDS beams for each type of lightweight aggregate are graphed in Figure 6.21.

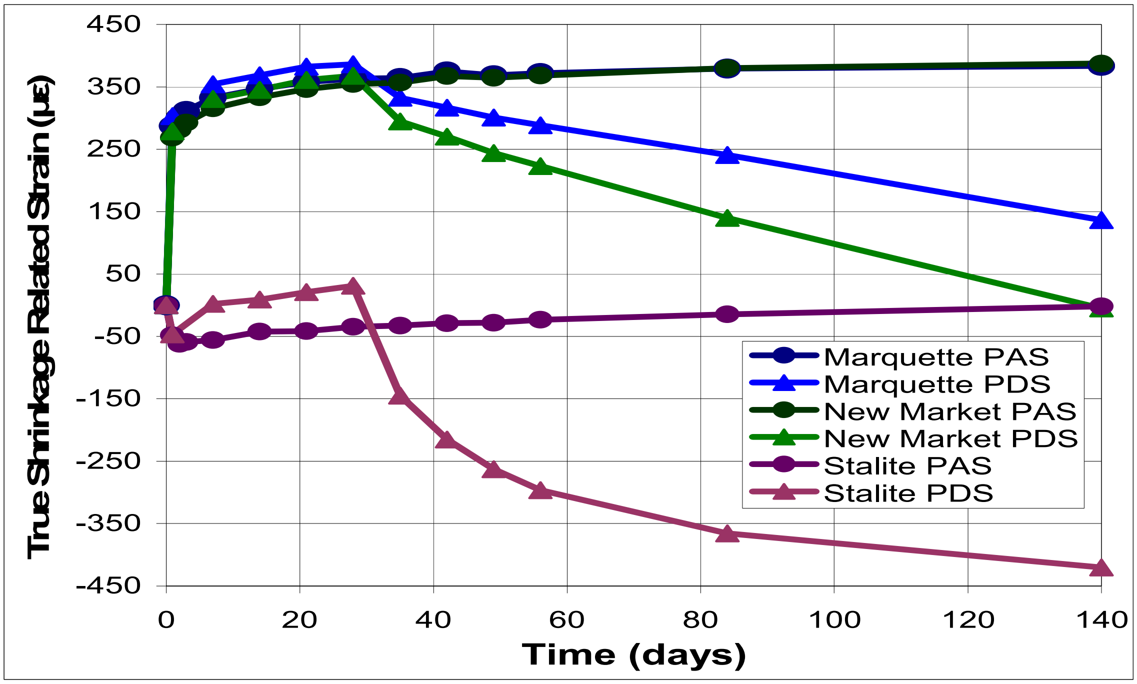

Figure 6.21. Average PAS and PDS shrinkage results for optimized prestressed mixtures for all three lightweight aggregates.

Since the prestressed Stalite concrete mixture initially behaved quite differently than all of the other five optimized bridge deck and prestressed concrete mixture, it was suspected that the initial gage readings were not recorded accurately. For this reason, the prestressed Stalite concrete mixture was repeated along with two additional concrete mixtures. Both additional concrete mixtures were variations of the Marquette bridge deck concrete mixture, the first replacing the normal sand with sand from a different source, and the second replacing the Marquette aggregate with normal-weight crushed gravel. The repeated prestressed Stalite concrete mixture was distinguished from the original prestressed Stalite concrete mixture by the #2. Fresh concrete properties for the three additional shrinkage mixtures is shown in Table 6.18. Shrinkage results for each of these concrete mixtures are shown in Figure 6.22, Figure 6.23, and Figure 6.24.

Table 6.18. Fresh concrete properties for three additional shrinkage mixtures.

	Volumetric Air Content (%)	Unit Weight (pcf)	Slump (in)
Prestressed Stalite #2	7.5	112.6	9.0
Sand Replacement	7.0	120.6	4.0
Gravel Replacement	6.0	142.1	5.5

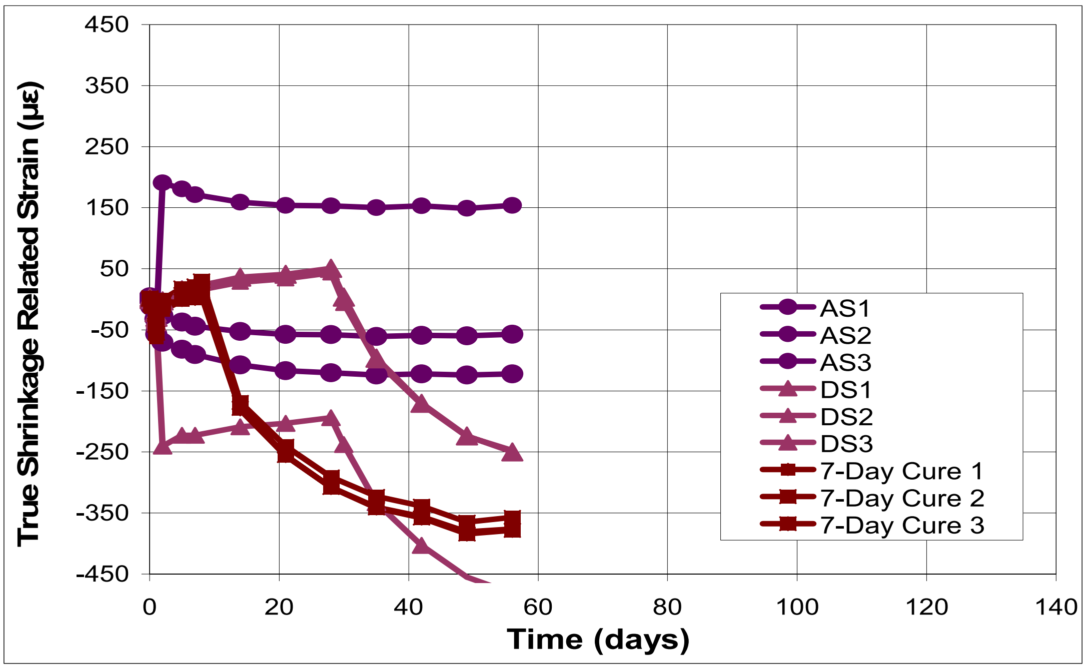

Figure 6.22. Shrinkage results for prestressed Stalite #2 concrete mixture.

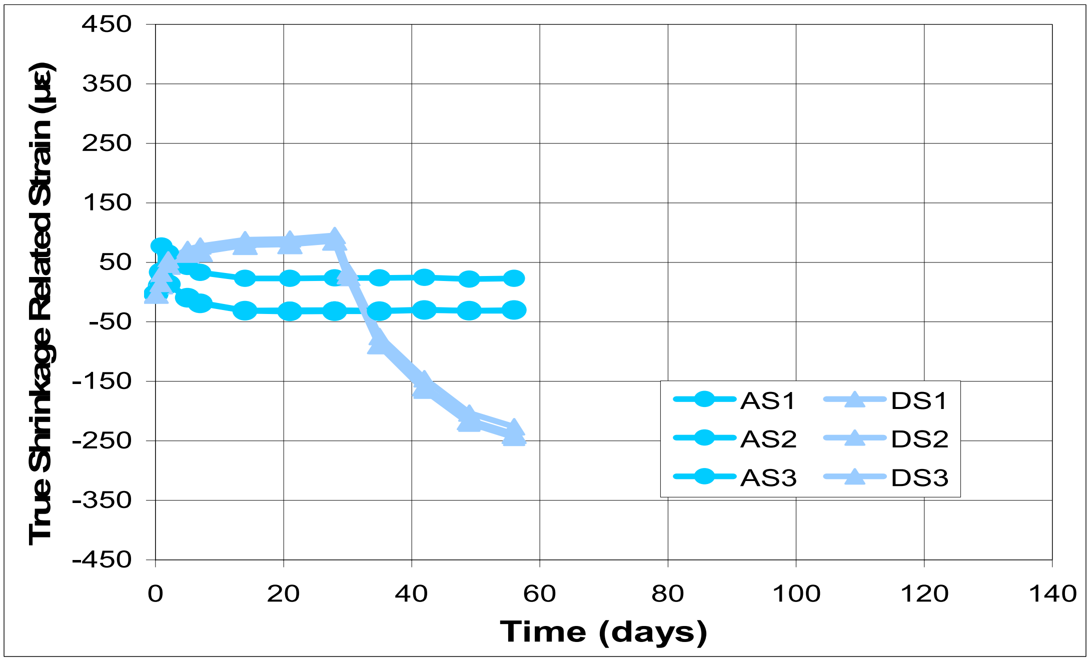

Figure 6.23. Shrinkage results for normal-weight gravel replacement concrete mixture.

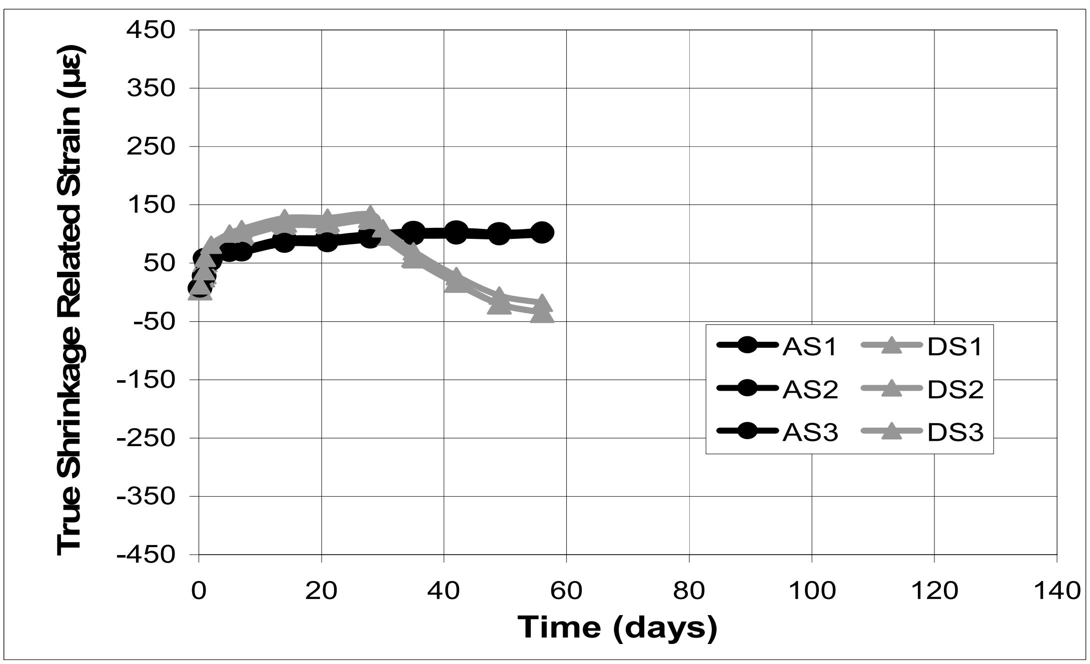

Figure 6.24. Shrinkage results for Nebraska sand replacement concrete mixture.

Shrinkage results for the prestressed concrete Stalite #2 concrete mixture appear to have two gages, from one AS beam and from on DS beam, that have a zero-shift occurring on the day 2 readings. The cause of this radical shift is unknown. It is interesting, however, to observe the trends of the gages after the reading shift. Both gages continue to follow similar length deformation trends as the other AS and DS beams. Furthermore, this mixture was repeated since it did not follow the initial expansion trend observed in all of the other five original shrinkage mixtures. The prestressed Stalite #2 concrete mixture also did not show initial expansion, but shrinkage. The difference between the original prestressed Stalite mixture and the prestressed Stalite #2 mixture is that the prestressed Stalite #2 AS beams, which are sealed from moisture transfer, continued to experience shrinkage whereas the original mixture started to expand slightly after day 2. Possible explanations for this are the amount of water that was absorbed in the internal aggregate structure. The prestressed Stalite mixture is the only concrete mixture containing Type III cement and the Stalite aggregate, which has the lowest absorption values. The Type III cement reacts faster to gain strength much quicker than Type I cement. It is believed that with the other shrinkage concrete mixtures, both the bridge deck and prestressed, the combination of the cement and absorbed water within the lightweight aggregate allowed adequate internal curing to prevent autogenous shrinkage, instead resulting in autogenous expansion. However, with the combination of the low absorption Stalite aggregate and the Type III cement, self-desiccation occurs at an increased rate and the moisture absorbed in the Stalite aggregate cannot transfer out of the internal aggregate pore structure fast enough to eliminate the autogenous shrinkage. In the case of the original prestressed Stalite mixture, since initial shrinkage followed by expansion was observed, it is believed that the amount of water absorbed in the aggregate was higher than in the #2 mixture. In this case, the moisture absorbed within the aggregate particle was unable to move out of the internal pore structure fast enough to prevent initial autogenous shrinkage, but once the cement hydration reaction slowed, after day two, there was adequate amount of absorbed moisture to mitigate continued shrinkage. With the prestressed Stalite #2 mixture, not quite enough moisture was present in the internal pore structure, and autogenous shrinkage continued to occur. The DS beams, on the other hand, demonstrated swelling once placed in the lime water storage tank as experienced by the other original shrinkage mixtures. For this prestressed Stalite #2, three additional beams were cast and the effect of drying shrinkage after only seven days of curing in the lime-water storage tank was evaluated. Results show similar trends to the 28-day cured DS beams, but the initial swelling that occurs while in the lime-water tank is less, so the following effect of shrinkage once out of the tank is greater.

The sand replacement and normal-weight gravel replacement concrete mixtures should be compared to the original optimized bridge deck Marquette concrete shrinkage results. The Nebraska sand replacement does seem to have an affect on shrinkage. The same length deformation trend occurred with both types of sand, but the magnitude of the initial expansion of the original sand source with the Marquette aggregate is much higher, about 300 $\mu\varepsilon$ compared to about 100 $\mu\varepsilon$. An explanation for this result is unknown, but further research should be conducted to determine the effects of different sand sources on shrinkage.

Results for the normal-weight gravel replacement mixture shows a zero-shift for the AS2 gage on day 1 readings after being demolded. This beam was accidentally dropped on day 1, and it is believed that the impact caused a small crack in the beam resulting in an expansion. The length deformation trend after this time is similar to that of the other AS beams. Therefore, the AS2 beam is believed to be following the same shrinkage and expansion trend of the other AS beams, but with a shift during the day 1 readings. Both the AS and DS beams experienced slight expansion in the first 24 hours, after that point, the DS beams were placed in the lime water storage tank and continued to swell. The AS beams were wrapped and sealed and then started to shrink. These results support the theory of internal curing from the absorbed moisture in lightweight aggregate since the normal-weight aggregate did not supply sufficient water to mitigate the autogenous shrinkage.

In all of the shrinkage concrete mixtures, the DS beams swelled when placed in the lime water storage tank, then experienced shrinkage when removed. In the case of the Marquette and New Market aggregates, the magnitude of overall shrinkage was lower since an increased amount of expansion was initially experienced. This decrease in overall shrinkage, especially at earlier ages, may be beneficial for lower cracking and increased durability performance. Furthermore, the effect of internal curing is observed in the AS beams for the optimized bridge deck mixtures for all three lightweight aggregate types. The internal curing affect is also observed for Marquette and New

Market aggregate in the prestressed concrete mixtures. The actual shrinkage results for all concrete mixtures are given in Appendix B -Shrinkage Results Summary. According to AASHTO M195, *Standard Specification for Lightweight Aggregates for Structural Concrete*, the maximum recommended drying shrinkage value for lightweight concrete is 0.07%. At 140 days, both the optimized bridge deck and optimized prestressed concrete mixtures, for all three lightweight aggregates, do not exceed this maximum recommended value.

Coefficient of Thermal Expansion

The coefficient of thermal expansion (CTE) was measured for both the optimized bridge deck and optimized prestressed concrete mixtures for each type of lightweight aggregate. One of the DS beams from the original shrinkage concrete mixtures was used to determine the CTE for each concrete mixture. The complete CTE test procedure is discussed in Chapter 5. Concrete property results for these beams are given in Table 6.16 and Table 6.17. The CTE results determined for each concrete mixture are shown in Table 6.19. The typical CTE value for normal-weight concrete is 5.5 x 10^{-6}/°F. CTE results for the lightweight concrete slightly lower than the typical value for normal-weight concrete due to the thermally-resistant nature of the lightweight aggregate.

Table 6.19. Coefficient of thermal expansion results.

	Coefficient of Thermal Expansion (in/in/°F)
Optimized Bridge Deck Concrete Mixtures	
Marquette	5.1 x 10^{-6}
New Market	5.1 x 10^{-6}
Stalite	5.2 x 10^{-6}
Optimized Prestressed Concrete Mixtures	
Marquette	5.3 x 10^{-6}
New Market	5.3 x 10^{-6}
Stalite	5.2 x 10^{-6}

CHAPTER 7 - Conclusions and Recommendations

Conclusions

The following conclusions can be made based on experimental results and observations collected during this study:

1. Lightweight aggregate concrete mixtures can be developed that are acceptable by KDOT specifications.
2. All three sources of lightweight aggregate were evaluated for gradation, absorption, and L.A. Abrasion, and were acceptable by KDOT standards.
3. All three lightweight aggregates, obtained from Buildex-Marquette, Buildex-New Market, and Stalite, were used to successfully create optimized bridge deck and optimized prestressed concrete mixtures.
4. Several mix design variables were evaluated to achieve the optimized concrete mix designs. For this project, the optimized bridge deck concrete contained 725 pcy Type I cement, a 0.38 water-to-cement ratio, and a 40% coarse, 60% fine aggregate ratio. The optimized prestressed concrete contained 725 pcy Type III cement, a 0.34 water-to-cement ratio, and a coarse-to-fine aggregate ratio to achieve a design unit weight of 118 pcf.
5. For the optimized bridge deck concrete mixture designs, the KDOT MA-2 gradation specification was met and several concrete properties in the plastic and hardened states were evaluated. Fresh concrete properties studied were slump, workability, and air content. Hardened concrete properties studied include compressive strength, tensile strength, and modulus of elasticity. All concrete properties evaluated satisfied KDOT requirements.
6. For the optimized prestressed concrete mixture designs, the KDOT gradation specification was not met in order to achieve a design unit weight of 118 pcf. However, several concrete properties in the fresh and hardened states were evaluated and found to satisfy KDOT requirements. Fresh concrete

properties studied were slump, workability, and air content. Hardened concrete properties studied include compressive strength, tensile strength, and modulus of elasticity.

7. All three lightweight aggregates produced concrete mixtures with results that are acceptable by KDOT specifications for the tests of freeze-thaw durability, permeability, and alkali-silica reactivity, with the exception of the Marquette aggregate and the freeze-thaw durability test, which should be evaluated again.
8. Successful optimized bridge deck concrete mixtures achieved adequate workability using only an air-entraining admixture.
9. Successful optimized prestressed concrete mixtures achieved adequate workability using an air-entraining admixture and a super-plasticizer admixture.
10. Autogenous shrinkage and drying shrinkage were measured for both the optimized bridge deck and optimized prestressed concrete for all three types of lightweight aggregate. Shrinkage values obtained are within the recommended AASHTO limit of 0.07%.
11. During the autogenous shrinkage evaluation, an obvious internal curing effect was observed due to the internal moisture content of the lightweight aggregate. With the Buildex-Marquette and Buildex-New Market aggregates, self-desiccation was prevented due to the internal curing effect and no autogenous shrinkage occurred.

Recommendations

The following are recommendations to KDOT about lightweight concrete in the state of Kansas:

1. The Buildex-Marquette and Buildex-New Market lightweight aggregates were used to develop successful optimized bridge deck concrete mixtures and could be used for KDOT bridge decks. The Stalite aggregate did

demonstrate satisfactory performance, but did not display any significant benefit to justify costs of shipping it to the state of Kansas.

2. The Buildex-Marquette and Buildex-New Market lightweight aggregates were used to develop successful optimized prestressed concrete mixtures and could be used for KDOT bridge girders. The Stalite aggregate did demonstrate satisfactory performance, but did not display any significant benefit to justify costs of shipping it to the state of Kansas.
3. Concrete unit weight values could be lowered by replacing the normal-weight sand with lightweight fine aggregate while still meeting KDOT gradation specification limits, as desired by KDOT personnel.
4. Modulus of elasticity values obtained for the lightweight concrete mixtures could likely be lowered if the coarse lightweight aggregate content was increased. However, changing the coarse-to-fine aggregate ratio would cause the gradation to fall outside of the KDOT specification limits. Another alternative for lowering the modulus of elasticity would be replacing the normal-weight sand with lightweight fine aggregate. More research should be conducted to determine the full effect of lightweight aggregate on the contact zone and resulting modulus of elasticity.
5. Lightweight aggregate should be pre-soaked before batching. A saturation period of at least seven days is recommended since the rate of absorption slows after this point. The moisture content of drained but wet lightweight aggregate can be successfully accounted for in the concrete mix design with good repeatability. Large-scale stock piles could be pre-soaked by a water sprinkling system.
6. The rollometer must be used to accurately determine air content of lightweight aggregate. If several consecutive batches of concrete from the same lightweight aggregate source and stockpile are being created, then the gravimetric method could be used to determine air content once the specific gravity of the lightweight aggregate has been calculated using the volumetric air content. Using the gravimetric method in this way would greatly expedite measuring the air content of consecutive concrete batches.

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Appendix A - Concrete Mixture Summary

The following tables show the comprehensive concrete mixture summaries for all of the concrete mixes created and tested during this project. Concrete mix design variables, fresh concrete properties, and compressive strength results are given for each mixture. Absence of data in these tables means that the data was not collected due to either an error in the testing procedure, the test was not applicable, or deemed unnecessary, to the particular concrete mixture.

Bridge Deck Concrete Mixture Summary

Date	Name	Aggregate Ratio	Aggregate Type	Air Entrainer (oz./100 lb cement)	w/c	Cement Content (lb/yd^3)	Slump (in)	% Air (Rollometer)	% Air (Gravimeteric)	Unit Weight (Gravametric) (pcf)	Average 7-day Strength (psi)	COV for 7-day strength (%)	Average 14-day Strength (psi)	COV for 14-day strength (%)	Average 28-day Strength (psi)	COV for 28-day strength (%)
10/10/2006	Marquette #1		Marq.	1.99	0.47	734	1.75	-		-	-					
10/30/2006	Marquette 10-17		Marq.	2.00	0.44	639	3.75	-		-	3303					
10/30/2006	Marquette 10-17 (2)		Marq.	2.00	0.40	639	0.5	-		-	4019					
12/14/2007	Marq 12-14 .4	producer's agg. Ratio	Marq.	2.00	0.40	639	3.5	-	7.2	117.8	3139				4322	
12/14/2007	Marq 12-14 .42	producer's agg. Ratio	Marq.	2.00	0.42	639	5.75	-	8.9	115.2	2719				3971	
12/15/2007	Marq 12-15 .44	producer's agg. Ratio	Marq.	2.00	0.44	639	7.5	-	16	107.2	1946				2738	
1/8/2007	Marq 1-8 .38	producer's agg. Ratio	Marq.	2.00	0.38	639	2.25	-	8.9	115.2	3213				4256	
1/8/2007	Marq .4 50-50	50-50	Marq.	1.52	0.40	639	3.25	-	10.9	112.5	3071				4279	
1/9/2007	Marq .42 50-50	50-50	Marq.	1.14	0.42	639	3.75	-	10.4	112.5	2589				3798	
1/9/2007	Marq .44 50-50	50-50	Marq.	0.95	0.44	639	7.75	-	10	112.5	2735				4078	
1/15/2007	Marq .4 40-60	40sand-60coarse	Marq.	1.52	0.40	639	2.5	9	8.9	109.9	2754				3549	
1/15/2007	Marq .42 40-60	40sand-60coarse	Marq.	1.14	0.42	639	4	-	11	107.2	2588				3350	
1/15/2007	Marq .44 40-60	40sand-60coarse	Marq.	0.95	0.44	639	5.5	-	13.2	104.5	2326				3199	
1/15/2007	Marq .4 60-40	60sand-40coarse	Marq.	1.52	0.40	639	3.25	9.5	10.4	117.8	2870				3984	
1/17/2007	Marq .42 60-40	60sand-40coarse	Marq.	1.14	0.42	639	6	-	9.9	117.8	2800				3787	
1/17/2007	Marq .44 60-40	60sand-40coarse	Marq.	0.95	0.44	639	7.5	-	7.2	120.5	2687				4004	
1/31/2007	Marq .4 700	60sand-40coarse	Marq.	1.22	0.40	700	5.5	8.25	10.3	117.8	3290	6.5			4470	1.3
1/31/2007	Marq .4 725	60sand-40coarse	Marq.	1.18	0.40	725	6.25	-	7.9	120.5	3520	2.2			4460	5.3
1/31/2007	Marq .4 750	60sand-40coarse	Marq.	1.14	0.40	750	6	-	10.1	117.8	3196	4.4			4200	3.9
1/31/2007	Marq .4 700 no air added	60sand-40coarse	Marq.	0.00	0.40	700	2.75	2	4	125.8	4440	3.1			5650	3.8
2/14/2007	Marq. .4 700 4ml	60sand-40coarse	Marq.	0.70	0.40	700	6.5	7.75	10.3	117.9	3170	0.9			4800	2
2/14/2007	Marq. .4 700 3ml	60sand-40coarse	Marq.	0.52	0.40	700	5.75	7.25	8.8	119.5	3480	3.1			4650	8.9
3/7/2007	Marq. .4 700 2.5ml	60sand-40coarse	Marq.	0.43	0.40	700	6.75	7	6.9	121.8	3320	2.9			5020	4.7
2/28/2007	Marq. .4 700 2ml	60sand-40coarse	Marq.	0.35	0.40	700	3.75	5	4.5	124.8	3840	4.7			5290	4.1
2/28/2007	Marq. .38 700 3ml	60sand-40coarse	Marq.	0.52	0.38	700	2.75	5.5	4.6	125.3	3890	4.9			5380	1.9
3/7/2007	Marq. .38 700 3.5ml	60sand-40coarse	Marq.	0.61	0.38	700	4.75	6.75	7.9	121.2	3730	7.2			5500	5.1
3/12/2007	Marq. .38 725 3.5ml	60sand-40coarse	Marq.	0.59	0.38	725	4.5	7	6.8	122.4	3530	8.9			4820	7.8
4/4/2007	Marq. .38 725 KDOT	60sand-40coarse	Marq.	0.52	0.38	725	5.5	6	7.2	121.9	3550	3.8	4260	2.8	5410	1.1
5/15/2007	LW Marq	42.8sand-57.2coarse	Marq.	0.60	0.36	675	1.5	4.5	3.4	118.6	4280	2.6			5820	4.3
7/16/2007	Marq. MOE	60sand-40coarse	Marq.	0.53	0.38	725	5	5	1.1	135.1	2740	11	4230	1	5050	5.9
10/22/2007	Marq. Shrinkage	60sand-40coarse	Marq.	0.59	0.38	725	5.25	8	12.3	120.8	3880	2.9			5600	1.8
3/12/2008	2KDOT rock (Marq. Replacement)	60sand-40coarse	Marq.	0.56	0.38	725	5.5	6	6.3	142.1	4540	3.8			6420	2.1
3/12/2008	2KDOT sand (NE. Replacement)	60sand-40coarse	Marq.	0.56	0.38	725	4	7	12.6	120.6	3640	3.4			5250	3.5

Table A.1. Marquette aggregate total bridge deck concrete mixture summary.

Date	Name	Aggregate Ratio	Aggregate Type	Air Entrainer (oz./100 lb cement)	w/c	Cement Content (lb/yd^3)	Slump (in)	% Air (Rollometer)	% Air (Gravimeteric)	Unit Weight (Gravametric) (pcf)	Average 7-day Strength (psi)	COV for 7-day strength (%)	Average 14-day Strength (psi)	COV for 14-day strength (%)	Average 28-day Strength (psi)	COV for 28-day strength (%)
10/10/2006	KC #1		KC	1.99	0.44	734	6.75	-		-						
10/17/2006	KC 10-17		KC	2.00	0.44	639	0.5	-		-	4706					
10/17/2006	KC 10-17 (2)		KC	2.00	0.40	639	0.5	-		-	3807					
12/14/2007	KC 12-14 .4	producer's agg. Ratio	KC	2.00	0.40	639	3.75	-	7.75	117.8	3103				4243	
12/14/2007	KC 12-14 .42	producer's agg. Ratio	KC	2.00	0.42	639	7	-	7.3	117.8	2584				3733	
12/15/2007	KC 12-15 .44	producer's agg. Ratio	KC	2.00	0.44	639	7	-	9.2	115.2	2175				3439	
1/8/2007	KC 1-8 .38	producer's agg. Ratio	KC	2.00	0.38	639	3.25	-	5.1	120.5	3433				4948	
1/9/2007	KC .4 50-50	50-50	KC	1.52	0.40	639	4	-	10.1	115.2	2581				4070	
1/9/2007	KC .42 50-50	50-50	KC	1.14	0.42	639	5.75	-	9.6	115.2	2918				4111	
1/9/2007	KC .44 50-50	50-50	KC	0.95	0.44	639	8.25	-	9.2	115.2	2466				3621	
1/10/2007	KC .4 40-60	40sand-60coarse	KC	1.52	0.40	639	3.5	-	8.4	112.5	2755				4045	
1/15/2007	KC .42 40-60	40sand-60coarse	KC	1.14	0.42	639	5.25	-	7.9	112.5	2729				3811	
1/15/2007	KC .44 40-60	40sand-60coarse	KC	0.95	0.44	639	8	-	15.2	104.5	2185				3280	
1/15/2007	KC .4 60-40	60sand-40coarse	KC	1.52	0.40	639	2.75	-	7.1	123.2	3223				4587	
1/17/2007	KC .42 60-40	60sand-40coarse	KC	1.14	0.42	639	6.25	-	8.9	120.5	2970				4027	
1/17/2007	KC .44 60-40	60sand-40coarse	KC	0.95	0.44	639	8	-	8.4	120.5	2655				3965	
2/5/2007	KC .4 700	60sand-40coarse	KC	1.04	0.40	700	8.75	-	11.4	117.8	2380	0.8			3570	4.5
2/5/2007	KC .4 750	60sand-40coarse	KC	0.97	0.40	750	7.75	-	8.8	120.5	3370	2.3			4950	3.2
2/5/2007	KC .4 700 no air added	60sand-40coarse	KC	0.00	0.40	700	2.5	1.5	3	128.5	4870	6.6			6830	9.3
2/14/2007	KC .4 700 4ml	60sand-40coarse	KC	0.70	0.40	700	6.25	8.5	6.9	123.1	3180	2.4			4980	1.2
2/14/2007	KC .4 700 3ml	60sand-40coarse	KC	0.52	0.40	700	6.75	7.5	5.3	125.1	3330	6.6			5360	3.4
2/28/2007	KC .4 700 2ml	60sand-40coarse	KC	0.35	0.40	700	3.5	5.5	4.8	125.7	4080	4.4			5620	0.7
3/7/2007	KC .4 700 2.5ml	60sand-40coarse	KC	0.43	0.40	700	6	8	6.5	123.6	3490	1.8			5710	3.6
2/28/2007	KC .38 700 3ml	60sand-40coarse	KC	0.52	0.38	700	2.25	5.5	4.5	126.7	4170	2.5			5880	5.3
3/7/2007	KC .38 700 4ml	60sand-40coarse	KC	0.70	0.38	700	4.5	9	7.3	123.2	3610	2.8			5230	2.9
3/12/2007	KC .38 725 3.5ml	60sand-40coarse	KC	0.59	0.38	725	3.25	6.75	6.8	126.2	3730	5.2			5900	4.5
4/4/2007	KC .38 725 KDOT	60sand-40coarse	KC	0.53	0.38	725	4.75	7.5	8	124.3	3820	5.7	4770	3.5	6210	2.8
5/15/2007	LW KC	31.6sand-68.4coarse	KC	0.60	0.37	675	1	6	4.8	117	3960	6.1			5860	3.8
7/17/2007	KC MOE	60sand-40coarse	KC	0.58	0.38	725	4	4.5	3.9	132.9	3330	5.2	4070	3.6	4910	6
10/22/2007	KC Shrinkage	60sand-40coarse	KC	0.59	0.38	725	5.25	7.5	13.6	121.1	3840	0.8			5730	1.4

Table A.2. New Market aggregate total bridge deck concrete mixture summary.

Date	Name	Aggregate Ratio	Aggregate Type	Air Entrainer (oz./100 lb cement)	w/c	Cement Content (lb/yd^3)	Slump (in)	% Air (Rollometer)	% Air (Gravimeteric)	Unit Weight (Gravametric) (pcf)	Average 7-day Strength (psi)	COV for 7-day strength (%)	Average 14-day Strength (psi)	COV for 14-day strength (%)	Average 28-day Strength (psi)	COV for 28-day strength (%)
10/10/2006	Staylite #1		Stalite	2.00	0.47	752	4.75	-		-	-					
10/30/2006	Staylite 10-30 (2)		Stalite	2.00	0.53	639	1.25	-		-	3405					
11/21/2006	Sta 11-21 .42		Stalite	0.00	0.42	639	5	-		117.8	3124					
12/14/2007	Sta 12-14 .4	producer's agg. Ratio	Stalite	2.00	0.40	639	6.25	-	9	115.2	2906				3841	
12/15/2007	Sta 12-15 .42	producer's agg. Ratio	Stalite	2.00	0.42	639	7.5	-	13.6	109.9	2120				3146	
12/15/2007	Sta 12-15 .44	producer's agg. Ratio	Stalite	2.00	0.44	639	8	-	10.5	112.5	1857				2772	
1/8/2007	Sta 1-8 .38	producer's agg. Ratio	Stalite	2.00	0.38	639	2.5	-	8.6	115.2	3036				4090	
1/10/2007	Stalite .4 50-50	50-50	Stalite	1.52	0.40	639	5.5	-	8.1	117.8	2632				3614	
1/9/2007	Stalite .42 50-50	50-50	Stalite	1.14	0.42	639	6.5	-	10	115.2	2810				3837	
1/10/2007	Stalite .44 50-50	50-50	Stalite	0.95	0.44	639	8.25	-	7.2	117.8	2535				3693	
1/10/2007	Stalite .4 40-60	40sand-60coarse	Stalite	1.52	0.40	639	4.75	-	11.2	109.9	2214				3383	
1/10/2007	Stalite .42 40-60	40sand-60coarse	Stalite	1.14	0.42	639	7	-	8.3	112.5	2321				3588	
1/10/2007	Stalite .44 40-60	40sand-60coarse	Stalite	0.95	0.44	639	7.75	-	7.9	112.5	2498				3452	
1/17/2007	Stalite .4 60-40	60sand-40coarse	Stalite	1.52	0.40	639	5	-	9.8	120.5	3108				4113	
1/17/2007	Stalite .42 60-40	60sand-40coarse	Stalite	1.14	0.42	639	8.5	-	11.7	117.8	2140				3107	
1/17/2007	Stalite .44 60-40	60sand-40coarse	Stalite	0.95	0.44	639	8.5	8.5	8.9	120.5	2766				3805	
2/5/2007	Stalite .4 700	60sand-40coarse	Stalite	1.04	0.40	700	7.75	-	9.3	120.5	2440	5.9			3640	12.6
2/5/2007	Stalite .4 750	60sand-40coarse	Stalite	0.97	0.40	750	8.75	-	6.8	123.2	2950	7			3990	9
2/5/2007	Stalite .4 700 no air added	60sand-40coarse	Stalite	0.00	0.40	700	5	1.5	3.2	128.5	4320	3.7			5770	0.7
2/28/2007	Stalite .4 700 2ml	60sand-40coarse	Stalite	0.35	0.40	700	6.75	4.75	5.7	124.8	3500	2.4			5300	2.3
3/12/2007	Stalite .38 725 3.5ml	60sand-40coarse	Stalite	0.59	0.38	725	7	7	7.3	123.3	3340	6.5			5370	4.7
4/11/2007	Stalite .38 725 KDOT	60sand-40coarse	Stalite	0.56	0.38	725	6.25	6.25	7.8	122.7	3580	2.9	4290	5.1	5220	3.2
5/15/2007	LW Sta.	31.6sand-68.4coarse	Stalite	0.61	0.36	675	2.5	5	4.2	117.7	4170	4.6			6310	5
7/17/2007	Sta. MOE	60sand-40coarse	Stalite	0.67	0.38	725	8	6.5	1.3	132.4	2900	4.7	3470	9.1	4160	7.6
10/22/2007	Stalite Shrinkage	60sand-40coarse	Stalite	0.59	0.38	725	6	6.5	11.1	120	3380	8.2			5240	4.2

Table A.3. Stalite aggregate total bridge deck concrete mixture summary.

Prestressed Concrete Mixture Summary

Date	Name	Trial #	Aggregate Ratio	Aggregate Type	Air Admixture (oz./100 lb cement)	Super Admixture Type	Superplacticizer (oz./100 lb cement)	w/c	Cement Content (lb/yd^3)	Slump (in)	% Air (Rollometer)	% Air (Gravimeteric)	Unit Weight (Gravametric) (pcf)
5/8/2007	Marq. .34 750	J-1	60sand-40coarse	Marq.	0.52	Daracem 100	7.3	0.34	750	0.5	2.5	2.3	124.8
5/9/2007	Marq. .32 750	J-2	60sand-40coarse	Marq.	0.67	Daracem 100	23.7	0.32	750	7.25	14	17.1	125.5
5/14/2007	Marq. 1 trial	sm-1	60sand-40coarse	Marq.	0.55	Daracem 100	12.8	0.34	750	0.5	-	2.9	128.6
5/14/2007	Marq. 2 trial	sm-2	60sand-40coarse	Marq.	0.55	Daracem 100	15.8	0.34	750	5.5	-	9	120.9
5/17/2007	Marq. III	3-1	60sand-40coarse	Marq.	0.64	Daracem 100	15.8	0.34	750	8	13	15.4	113.9
5/21/2007	4Marq. #1	4-1	60sand-40coarse	Marq.	0.58	Daracem 100	15.8	0.34	750	0.25	-	4.2	126.9
5/21/2007	4Marq. #2	4-2	60sand-40coarse	Marq.	0.58	Daracem 100	17.7	0.34	750	4.25	-	10.3	119.4
5/21/2007	4Marq. #3	4-3	60sand-40coarse	Marq.	0.55	Daracem 100	18.9	0.34	750	8.5	-	19.3	109.9
5/21/2007	4Marq. #4	4-4	60sand-40coarse	Marq.	0.52	Daracem 100	18.3	0.34	750	9.25	-	16.8	112.4
5/22/2007	5-1 Marq.	5-1	60sand-40coarse	Marq.	0.39	Daracem 100	17.7	0.34	750	6.25	9	12.2	117.3
5/22/2007	5-2 Marq.	5-2	60sand-40coarse	Marq.	0.34	Daracem 100	17.7	0.34	750	6.5	-	12.4	115.3
5/22/2007	5-3 Marq.	5-3	60sand-40coarse	Marq.	0.28	Daracem 100	17.7	0.34	750	8	-	14	115.3
5/30/2007	6-1 Marq.	6-1	60sand-40coarse	Marq.	0.00	Daracem 100	17.7	0.34	750	4	5.75	7.6	122.6
6/4/2007	9-1 Marq.	9-1	60sand-40coarse	Marq.	0.00	AdvaCast 530	7.3	0.34	750	8.5	6.5	6.6	123.8
6/6/2007	10-1 Marq.	10-1	60sand-40coarse	Marq.	0.12	AdvaCast 530	6.0	0.34	750	5.5	5.25	4.7	126.2
6/7/2007	11-1 Marq.	11-1	60sand-40coarse	Marq.	0.15	AdvaCast 530	5.5	0.34	750	2.75	4	2.9	128.6
6/14/2007	12-1 Marq.	12-1	60sand-40coarse	Marq.	0.15	AdvaCast 530	6.3	0.34	725	5.75	6	5.7	125.0
6/19/2007	13-1 Marq.	13-1	60sand-40coarse	Marq.	0.15	AdvaCast 530	6.5	0.34	725	6.5	6.5	5.9	124.8
6/20/2007	14-1 Marq. NO SSD	14-1	60sand-40coarse	Marq.	0.15	AdvaCast 530	6.5	0.34	725	3	5	7.8	126.9
6/26/2007	15-1 Marq. LBPT trial	15-1	60sand-40coarse	Marq.	0.10	AdvaCast 530	6.0	0.34	725	1	estimated 2-3%	-	-
7/3/2007	16-1 Marq. 3"	16-1	47.7sand-52.3coarse	Marq.	0.20	AdvaCast 530	6.2	0.34	725	4	5.75	5	119.8
7/3/2007	16-2 Marq. 9"	16-2	47.7sand-52.3coarse	Marq.	0.00	AdvaCast 530	8.2	0.34	725	8	7	6.3	118.2
7/26/2007	LBPT Marq. 3"	try #1 LBPT	47.7sand-52.3coarse	Marq.	0.25	AdvaCast 530	6.2	0.34	725	3" @pour	4	2.1	123.4
7/31/2007	LBPT Marq. 3" #2	LBPT	47.7sand-52.3coarse	Marq.	0.30	AdvaCast 530	5.9	0.34	725	3" @pour	3.5	0.6	125.4
8/13/2007	LBPT Marq. 9"	LBPT	47.7sand-52.3coarse	Marq.	0.00	AdvaCast 530	6.7	0.34	725	9" @pour	2.5	0.3	125.8
10/16/2007	Marq. 3" creep #1	trial 1 creep	47.7sand-52.3coarse	Marq.	0.15	AdvaCast 530	6.5	0.34	725	3" @pour	6.5	5.7	118.9
10/19/2007	Marq. 3" creep #2	trial 2 creep	47.7sand-52.3coarse	Marq.	0.15	AdvaCast 530	6.3	0.34	725	3" @pour	5	3.8	121.3
11/6/2007	Marq. 3" creep #3	creep	47.7sand-52.3coarse	Marq.	0.15	AdvaCast 530	6.5	0.34	725	3" @pour	8	8.6	115.6
10/23/2007	Marq. shrinkage	shrinkage	47.7sand-52.3coarse	Marq.	0.15	AdvaCast 530	6.4	0.34	725	7	8	7.2	117.2
1/14/2008	Marq. 9" beam	beams	47.7sand-52.3coarse	Marq.	0.30	AdvaCast 530	6.1	0.34	725	9" @pour	5.5	2.9	122.4
1/22/2008	Marq. 3" beam	beams	47.7sand-52.3coarse	Marq.	0.35	AdvaCast 530	5.5	0.34	725	3" @pour	4	2.9	122.4

Table A.4. Marquette aggregate prestressed fresh concrete mixture summary.

Date	Name	Trial #	Average 16-hour Heated Strength (psi)	COV for 16-hour strength (%)	Average 16-hour Un-heated Strength (psi)	COV for 16-hour strength (%)	Average 24-hour Strength (psi)	COV for 24-hour strength (%)	Average test Strength (psi)	COV for test strength (%)	Average 7-day Strength (psi)	COV for 7-day strength (%)	Average 28-day Strength (psi)	COV for 28-day strength (%)
5/8/2007	Marq. .34 750	J-1	4880	2.4			5360	1.1			6560	0.6		
5/9/2007	Marq. .32 750	J-2	1370	3.6										
5/14/2007	Marq. 1 trial	sm-1	-	-										
5/14/2007	Marq. 2 trial	sm-2	-	-										
5/17/2007	Marq. III	3-1	-	-										
5/21/2007	4Marq. #1	4-1	-											
5/21/2007	4Marq. #2	4-2	-											
5/21/2007	4Marq. #3	4-3	-											
5/21/2007	4Marq. #4	4-4	-											
5/22/2007	5-1 Marq.	5-1	-											
5/22/2007	5-2 Marq.	5-2	-											
5/22/2007	5-3 Marq.	5-3	-											
5/30/2007	6-1 Marq.	6-1	5170	4.6	3720	3.2								
6/4/2007	9-1 Marq.	9-1	6350	2.8	5510	0.9	6380	2.6			7030	2.5		
6/6/2007	10-1 Marq.	10-1	6210	0.6	5690	1.3								
6/7/2007	11-1 Marq.	11-1	5990	1.4	5480	2.9								
6/14/2007	12-1 Marq.	12-1	-		4430	3.8								
6/19/2007	13-1 Marq.	13-1	5470	4.5	4670	5.9					6570	0.9		
6/20/2007	14-1 Marq. NO SSD	14-1	5380	1.9	-						6530	2.2	7090	4.8
6/26/2007	15-1 Marq. LBPT trial	15-1	-		-									
7/3/2007	16-1 Marq. 3"	16-1	5610	4.4					6420 (2-day)	1.9			7160	0.4
7/3/2007	16-2 Marq. 9"	16-2	5640	4.7					6250 (2-day)	3.1			7070	2.6
7/26/2007	LBPT Marq. 3"	try #1 LBPT					6050	1	6810	0.5				
7/31/2007	LBPT Marq. 3" #2	LBPT							5110	1.5				
8/13/2007	LBPT Marq. 9"	LBPT							5220	0.2				
10/16/2007	Marq. 3" creep #1	trial 1 creep												
10/19/2007	Marq. 3" creep #2	trial 2 creep												
11/6/2007	Marq. 3" creep #3	creep											7700	2.8
10/23/2007	Marq. shrinkage	shrinkage					5480	2.9					7130	3.7
1/14/2008	Marq. 9" beam	beams					5610	3.5					7260	0.6
1/22/2008	Marq. 3" beam	beams					6180	1.6					7930	1.5

Table A.5. Marquette aggregate prestressed concrete strength summary.

Date	Name	Trial #	Aggregate Ratio	Aggregate Type	Air Admixture (oz./100 lb cement)	Super Admixture Type	Superplacticizer (oz./100 lb cement)	w/c	Cement Content (lb/yd^3)	Slump (in)	% Air (Rollometer)	% Air (Gravimeteric)	Unit Weight (Gravametric) (pcf)
5/8/2007	KC .34 750	J-1	60sand-40coarse	KC	0.55	Daracem 100	10.0	0.34	750	2.25	5.25	3.7	125.9
5/17/2007	KC III	3-1	60sand-40coarse	KC	0.52	Daracem 100	12.2	0.34	750	1.75	-	3.5	131.3
5/17/2007	KC III 2	3-2	60sand-40coarse	KC	0.58	Daracem 100	14.0	0.34	750	4.25	5.75	4.7	129.7
6/1/2007	8-1 KC	8-1	*	KC	0.58	Daracem 100	14.0	0.34	750	5.5	9.25	16.3	116.0
6/4/2007	9-1 KC	9-1	60sand-40coarse	KC	0.52	Daracem 100	14.0	0.34	750	0.5	-	8.7	124.6
6/6/2007	10-1 KC	10-1	60sand-40coarse	KC	0.15	AdvaCast 530	6.0	0.34	750	2.5	4	10.3	122.6
6/6/2007	10-2 KC	10-2	60sand-40coarse	KC	0.17	AdvaCast 530	6.1	0.34	750	5	6	12.7	119.8
6/7/2007	11-1 KC	11-1	60sand-40coarse	KC	0.18	AdvaCast 530	6.4	0.34	750	2.5	5	10.9	122.0
6/14/2007	12-1 KC	12-1	60sand-40coarse	KC	0.18	AdvaCast 530	6.7	0.34	725	9.25	-	15.5	116.8
6/19/2007	13-1 KC	13-1	60sand-40coarse	KC	0.15	AdvaCast 530	6.9	0.34	725	4	6.5	12	120.8
6/20/2007	14-1 KC NO SSD	14-1	60sand-40coarse	KC	0.15	AdvaCast 530	6.9	0.34	725	3	-	13	122.5
10/19/2007	KC 3" creep #1	trial 1 creep	44sand-56coarse	KC	0.15	AdvaCast 530	6.9	0.34	725	3" @pour	8	8.25	116.0
11/6/2007	KC 3" creep #2	creep	44sand-56coarse	KC	0.15	AdvaCast 530	6.1	0.34	725	3" @pour	7.5	5.1	119.7
8/15/2007	LBPT KC 3"	LBPT	44sand-56coarse	KC	0.30	AdvaCast 530	5.4	0.34	725	3" @pour	3.75	4.1	120.9
8/21/2007	LBPT KC 9"	LBPT	44sand-56coarse	KC	0.30	AdvaCast 530	6.4	0.34	725	9" @pour	5	9.1	115.0
10/23/2007	KC shrinkage	shrinkage	44sand-56coarse	KC	0.14	AdvaCast 530	6.7	0.34	725	7	8	8.1	116.2
1/28/2008	KC 9" beam	beams	44sand-56coarse	KC	0.35	AdvaCast 530	6.1	0.34	725	9" @pour	6	9.1	115.0
3/24/2008	KC 3" beam	beams	44sand-56coarse	KC	0.37	AdvaCast 530	5.4	0.34	725	3" @pour	4	2.5	122.9

Table A.6. New Market aggregate prestressed fresh concrete mixture summary.

Date	Name	Trial #	Average 16-hour Heated Strength (psi)	COV for 16-hour strength (%)	Average 16-hour Un-heated Strength (psi)	COV for 16-hour strength (%)	Average 24-hour Strength (psi)	COV for 24-hour strength (%)	Average test Strength (psi)	COV for test strength (%)	Average 7-day Strength (psi)	COV for 7-day strength (%)	Average 28-day Strength (psi)	COV for 28-day strength (%)
5/8/2007	KC .34 750	J-1	4110	7.7			5130	3.4			6790	-		
5/17/2007	KC III	3-1	-	-										
5/17/2007	KC III 2	3-2	5510	1.8			6220	1						
6/1/2007	8-1 KC	8-1	3830	5.9	2700	3	4070	5.3						
6/4/2007	9-1 KC	9-1	-											
6/6/2007	10-1 KC	10-1	-											
6/6/2007	10-2 KC	10-2	5520	0.7	5040	2.4								
6/7/2007	11-1 KC	11-1	5520	5.7	5640	4.2								
6/14/2007	12-1 KC	12-1	-		-									
6/19/2007	13-1 KC	13-1	5630	0.9	5020	2.5					6410	5.6		
6/20/2007	14-1 KC NO SSD	14-1	5660	3.5	-						6710	1.5	7260	1
10/19/2007	KC 3" creep #1	trial 1 creep												
11/6/2007	KC 3" creep #2	creep											8140	3.3
8/15/2007	LBPT KC 3"	LBPT							5250	1.3				
8/21/2007	LBPT KC 9"	LBPT							5250	1.7				
10/23/2007	KC shrinkage	shrinkage					5500	2.1					7310	3.4
1/28/2008	KC 9" beam	beams					5900	5.1					7390	9.1
3/24/2008	KC 3" beam	beams					5530	4.2					7870	6

Table A.7. New Market aggregate prestressed concrete strength summary.

Date	Name	Trial #	Aggregate Ratio	Aggregate Type	Air Admixture (oz./100 lb cement)	Super Admixture Type	Superplacticizer (oz./100 lb cement)	w/c	Cement Content (lb/yd^3)	Slump (in)	% Air (Rollometer)	% Air (Gravimeteric)	Unit Weight (Gravametric) (pcf)
5/8/2007	Stalite .42 639	J-1	60sand-40coarse	Stalite	0.64	Daracem 100	8.6	0.42	639	7	6.5	8.2	125.7
5/9/2007	Stalite .32 750	J-2	60sand-40coarse	Stalite	0.67	Daracem 100	15.6	0.32	750	0.5	2	3.6	128.2
5/14/2007	Stalite .34 750	sm-1	60sand-40coarse	Stalite	0.49	Daracem 100	15.8	0.34	750	1	-	5.5	126.7
5/14/2007	Stalite .34 750	sm-2	60sand-40coarse	Stalite	0.49	Daracem 100	17.5	0.34	750	6.25	-	10.1	121.2
5/14/2007	Stalite .34 750	sm-3	60sand-40coarse	Stalite	0.43	Daracem 100	17.5	0.34	750	5.5	-	11.8	119.2
5/17/2007	Sta. III	3-1	60sand-40coarse	Stalite	0.43	Daracem 100	17.5	0.34	750	9	-	17.9	112.7
5/30/2007	6-1 Sta.	6-1	60sand-40coarse	Stalite	0.58	Daracem 100	15.8	0.34	750	3.5	8.5	9.5	121.8
5/30/2007	6-2 Sta.	6-2	60sand-40coarse	Stalite	0.52	Daracem 100	17.7	0.34	750	9	-	17	113.6
5/31/2007	7-1 Sta.	7-1	60sand-40coarse	Stalite	0.52	Daracem 100	16.8	0.34	750	1.25	-	8.1	123.6
5/31/2007	7-2 Sta.	7-2	60sand-40coarse	Stalite	0.43	Daracem 100	17.7	0.34	750	8.25	-	17	113.6
6/1/2007	8-1 Sta.	8-1	60sand-40coarse	Stalite	0.46	Daracem 100	17.2	0.34	750	5.75	-	16.8	113.7
6/1/2007	8-2 Sta.	8-2	60sand-40coarse	Stalite	0.33	Daracem 100	17.2	0.34	750	3.5	6.75	10.4	120.8
6/4/2007	9-1 Sta.	9-1	60sand-40coarse	Stalite	0.33	Daracem 100	17.2	0.34	750	-	16	114.7	-
6/7/2007	11-1 Sta.	11-1	60sand-40coarse	Stalite	0.15	AdvaCast 530	6.4	0.34	750	4	5	7.5	124.2
6/14/2007	12-1 Stalite	12-1	60sand-40coarse	Stalite	0.15	AdvaCast 530	6.9	0.34	725	7.25	6.75	9.1	122.4
6/19/2007	13-1 Stalite	13-1	60sand-40coarse	Stalite	0.15	AdvaCast 530	6.9	0.34	725	6.5	6.5	8	123.7
6/20/2007	14-1 Stalite NO SSD	14-1	60sand-40coarse	Stalite	0.15	AdvaCast 530	6.9	0.34	725	6	6.5	9.5	122.9
10/19/2007	Sta. 3" creep #1	trial 1 creep	43.6sand-56.4coarse	Stalite	0.15	AdvaCast 530	6.6	0.34	725	3" @pour	6	7.2	117.2
11/6/2007	Sta. 3" creep #2	creep	43.6sand-56.4coarse	Stalite	0.15	AdvaCast 530	5.7	0.34	725	3" @pour	5	5.4	119.3
Oct. 2007?	LBPT Sta. 3"	LBPT	43.6sand-56.4coarse	Stalite	0.34	AdvaCast 530	4.9	0.34	725	3" @pour	3	6.3	118.2
11/20/2007	LBPT Sta. 9"	LBPT	43.6sand-56.4coarse	Stalite	0.30	AdvaCast 530	6.4	0.34	725	9" @pour	-	12.3	111.6
10/23/2007	Stalite shrinkage	shrinkage	43.6sand-56.4coarse	Stalite	0.15	AdvaCast 530	6.8	0.34	725	9	6	-	117.0
3/10/2008	Stalite 9" beam	beams	43.6sand-56.4coarse	Stalite	0.32	AdvaCast 530	6.2	0.34	725	9" @pour	7	7.7	116.6
3/31/2008	Stalite 3" beam	beams	43.6sand-56.4coarse	Stalite	0.39	AdvaCast 530	5.0	0.34	725	3" @pour	3.5	4.1	120.9
3/12/2008	PS Stalite shrinkage #2	shrinkage	43.6sand-56.4coarse	Stalite	0.15	AdvaCast 530	6.3	0.34	725	9	7.5	11.3	112.6

Table A.8. Stalite aggregate prestressed fresh concrete mixture summary.

Date	Name	Trial #	Average 16-hour Heated Strength (psi)	COV for 16-hour strength (%)	Average 16-hour Un-heated Strength (psi)	COV for 16-hour strength (%)	Average 24-hour Strength (psi)	COV for 24-hour strength (%)	Average test Strength (psi)	COV for test strength (%)	Average 7-day Strength (psi)	COV for 7-day strength (%)	Average 28-day Strength (psi)	COV for 28-day strength (%)
5/8/2007	Stalite .42 639	J-1	2410	7.6			3380	1			4660	0.4		
5/9/2007	Stalite .32 750	J-2	5550	1.4			6440	3.2			7720	-		
5/14/2007	Stalite .34 750	sm-1	-	-										
5/14/2007	Stalite .34 750	sm-2	-	-										
5/14/2007	Stalite .34 750	sm-3	-	-										
5/17/2007	Sta. III	3-1	-	-										
5/30/2007	6-1 Sta.	6-1	4820	2.5	3730	4.3								
5/30/2007	6-2 Sta.	6-2	-											
5/31/2007	7-1 Sta.	7-1	-											
5/31/2007	7-2 Sta.	7-2	-											
6/1/2007	8-1 Sta.	8-1	-											
6/1/2007	8-2 Sta.	8-2	5530	3.1	4440	2.5	5860	3.9						
6/4/2007	9-1 Sta.	9-1												
6/7/2007	11-1 Sta.	11-1	6580	3.3	6000	1.6								
6/14/2007	12-1 Stalite	12-1	-		5660	1.6								
6/19/2007	13-1 Stalite	13-1	6460	1.3	5710	2.4					7240	7		
6/20/2007	14-1 Stalite NO SSD	14-1	6720	4.8	-						7790	2.5	8360	2.6
10/19/2007	Sta. 3" creep #1	trial 1 creep												
11/6/2007	Sta. 3" creep #2	creep											9260	2.8
Oct. 2007?	LBPT Sta. 3"	LBPT							4950	3.7				
11/20/2007	LBPT Sta. 9"	LBPT							5010	3.9				
10/23/2007	Stalite shrinkage	shrinkage					5880	2.8					8060	2.1
3/10/2008	Stalite 9" beam	beams											8230	2.8
3/31/2008	Stalite 3" beam	beams					5830	2.1					8010	2.9
3/12/2008	PS Stalite shrinkage #2	shrinkage					5980	0.8					7880	6.9

Table A.9. Stalite aggregate prestressed concrete strength summary.

Appendix B - Shrinkage Results Summary

The following tables show the actual length deformation data recorded during the shrinkage experiment discussed in Chapters 4 and 5. For each concrete mixture there were three beams. Autogenous shrinkage beams are designated as AS and drying shrinkage beams are designated DS. For these length deformation values, positive numbers indicate expansion and negative values indicate shrinkage.

Optimized Bridge Deck Concrete Mixtures

Table B.1. Marquette bridge deck concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.21	10.6	11.9	10.8	20.8	9.5	10.3
0.96	27.8	32.7	31.1	40.6	28.9	32.8
1.13	32.6	38.1	36.7	47.5	33.2	37.5
1.27	59.7	62.7	60.4	71.1	57.2	61.8
2.06	258.1	263.1	262.4	-	-	-
3.00	261.9	267.6	266.5	-	-	-
4.00	268.2	273.5	272.9	-	-	-
7.00	286.4	292.2	291.6	309.6	281.1	302.5
14.00	300.6	307.6	304.7	319.9	292.7	314.2
22.00	308.4	314.3	310.6	331.9	305.0	326.1
28.00	315.4	322.3	318.5	338.7	312.5	334.0
35.00	317.0	324.5	321.4	258.6	225.3	249.8
42.00	316.0	326.0	321.4	224.9	188.9	219.4
49.00	313.1	325.9	319.4	193.1	152.1	182.2
56.00	320.2	334.1	327.6	166.1	121.1	152.0
84.00	324.5	340.5	323.2	46.8	-6.7	35.3
140.00	338.1	351.7	320.2	-193.2	-252.7	-205.9

Table B.2. New Market bridge deck concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.71	19.9	25.7	25.6	26.3	35.2	18.5
0.83	28.1	32.5	31.8	33.3	42.2	25.6
1.04	238.1	256.5	258.9	256.1	269.7	252.9
1.81	237.8	253.8	252.4	-	-	-
3.00	242.7	258.3	257.0	-	-	-
4.00	247.4	262.6	262.3	-	-	-
7.00	265.4	281.5	281.2	290.8	303.4	286.8
14.00	280.3	298.3	296.6	302.4	313.9	297.7
22.00	289.9	308.3	307.9	315.1	328.6	312.3
28.00	302.6	315.2	316.1	323.0	337.1	319.6
35.00	304.9	318.0	319.0	242.0	253.4	238.7
42.00	305.9	318.2	318.9	193.9	205.6	194.0
49.00	305.8	317.6	319.1	150.9	161.5	148.0
56.00	312.8	361.3	326.9	114.9	125.9	102.8
84.00	317.6	330.6	332.0	-44.2	-30.7	-43.2
140.00	330.2	344.4	349.8	-239.3	-205.8	-213.0

Table B.3. Stalite bridge deck concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.67	23.4	25.5	30.3	19.5	16.7	16.8
0.79	29.9	32.5	38.0	26.4	22.8	22.9
1.00	45.8	49.3	55.3	45.6	42.9	42.1
1.79	232.5	243.9	255.8	-	-	-
3.00	230.3	241.5	253.0	-	-	-
4.00	228.3	238.5	251.5	-	-	-
7.00	227.4	238.9	251.0	287.4	286.0	282.7
14.00	220.1	231.5	242.7	295.6	294.7	291.9
22.00	219.0	230.3	241.2	306.8	305.4	302.1
28.00	221.6	233.2	244.6	314.6	312.2	309.8
35.00	222.4	235.3	245.9	148.9	149.6	145.0
42.00	224.6	238.8	250.1	67.8	61.4	69.8
49.00	225.2	240.2	250.2	6.3	-2.1	7.3
56.00	224.5	240.5	249.5	-7.3	-45.5	-34.8
84.00	228.9	251.6	252.0	-155.6	-153.9	-152.5
140.00	213.4	262.9	236.8	-271.5	-268.1	-275.4

Table B.4. Marquette prestressed concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.96	286.8	282.2	293.1	294.8	301.1	313.2
2.00	299.2	296.4	308.1	-	-	-
3.00	308.7	306.2	318.2	-	-	-
7.00	329.3	328.7	339.2	345.2	352.2	364.4
14.00	343.7	343.4	353.5	361.0	364.5	379.6
21.00	354.2	354.2	363.7	374.4	381.1	391.7
28.00	359.7	358.8	370.5	378.5	386.5	393.2
35.00	360.7	360.0	372.2	323.3	334.2	339.9
42.00	372.0	368.2	383.1	310.4	317.5	321.9
49.00	368.3	360.8	376.1	293.3	301.3	307.5
56.00	372.6	361.9	381.0	280.6	289.5	295.0
84.00	379.3	363.0	395.2	237.5	240.0	244.5
140.00	388.2	358.5	403.3	142.5	124.8	142.1

Table B.5. New Market prestressed concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.90	267.6	270.5	268.2	285.7	275.0	275.3
2.00	279.6	284.0	280.4	-	-	-
3.00	291.3	295.3	291.2	-	-	-
7.00	313.9	317.7	314.4	336.7	327.6	326.6
14.00	332.4	335.3	331.7	351.7	342.5	341.6
21.00	344.4	348.4	346.0	368.1	357.1	357.5
28.00	352.6	356.5	354.4	375.2	363.2	364.3
35.00	354.7	359.2	356.4	301.6	289.8	291.9
42.00	365.7	369.7	366.4	277.6	265.6	267.6
49.00	362.5	368.8	362.1	251.5	240.1	240.3
56.00	366.2	373.2	365.2	229.4	219.3	219.9
84.00	379.2	387.0	373.6	146.9	139.1	133.0
140.00	389.9	393.2	378.7	-7.1	-1.2	-7.8

Table B.6. Stalite prestressed concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.90	-26.1	-66.1	-52.0	-43.1	-43.6	-52.7
2.00	-48.2	-77.0	-61.0	-	-	-
3.00	-45.8	-74.0	-56.7	-	-	-
7.00	-32.7	-77.1	-57.7	10.8	3.6	-8.5
14.00	-9.4	-51.4	-66.4	18.2	11.2	-2.5
21.00	-1.0	-45.9	-77.7	31.7	24.7	6.9
28.00	9.2	-36.3	-77.3	42.0	34.3	17.1
35.00	13.6	-32.4	-79.5	-136.2	-150.0	-148.1
42.00	22.3	-25.2	-83.6	-210.0	-221.1	-214.5
49.00	23.6	-23.2	-84.3	-261.0	-267.9	-259.8
56.00	30.1	-17.2	-83.7	-297.3	-299.8	-291.9
84.00	44.1	-1.2	-86.5	-369.0	-370.6	-357.6
140.00	54.7	19.9	-80.3	-424.5	-426.4	-410.1

Additional Shrinkage Concrete Mixtures

Table B.7. Stalite prestressed #2 concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)	7-Day Cure 1 (με)	7-Day Cure 2 (με)	7-Day Cure 3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.05	3.9	-0.2	0.4	1.4	1.9	5.0	0.5	0.7	2.5
0.24	-7.8	-12.1	-6.8	-0.6	-4.1	0.7	-13.5	-6.4	-8.4
0.91	-14.6	-32.0	-52.3	-22.2	-30.5	-34.6	-51.6	-24.7	-42.7
1.04	-16.3	-57.0	-40.2	-29.6	-40.8	-38.5	-62.8	-37.7	-55.5
2.00	-26.9	-70.4	190.8	-2.7	-3.6	-240.4	-18.2	-0.5	-10.0
5.00	-37.3	-81.9	180.9	15.8	13.8	-222.5	-1.8	18.5	8.0
7.00	-44.1	-90.2	171.5	21.0	13.7	-222.5	0.0	22.2	7.3
8.00	0.0	0.0	0.0	0.0	0.0	0.0	6.2	30.1	14.0
14.00	-52.5	-107.4	159.0	36.4	28.1	-208.7	-180.6	-168.9	-178.4
21.00	-57.6	-116.8	153.8	41.0	33.6	-203.1	-256.3	-240.7	-255.8
28.00	-58.0	-120.1	152.9	50.7	43.3	-193.8	-307.8	-290.9	-308.3
35.00	-60.9	-123.9	150.0	-96.2	-100.7	-333.0	-340.4	-322.6	-341.9
42.00	-58.9	-122.0	152.9	-170.1	-171.7	-403.1	-355.8	-338.9	-358.2
49.00	-59.9	-123.9	149.1	-223.9	-224.5	-454.9	-380.7	-364.8	-384.1
56.00	-57.0	-122.0	153.9	-249.8	-247.6	-481.8	-375.9	-357.1	-378.3

Table B.8. Gravel replacement concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.07	-3.2	-1.2	-3.3	-3.3	-2.2	-3.9
0.78	10.7	13.0	8.2	12.8	13.8	14.8
0.94	33.4	77.2	28.0	27.2	29.7	31.8
2.00	13.2	65.4	14.4	45.7	54.5	46.8
5.00	-9.7	42.9	-9.7	61.7	71.7	63.8
7.00	-19.1	33.0	-17.9	67.2	77.6	69.5
14.00	-31.5	23.0	-30.8	78.9	87.9	82.3
21.00	-32.7	22.9	-30.5	80.4	89.4	83.0
28.00	-32.2	23.8	-30.2	85.9	95.4	87.9
35.00	-32.2	23.8	-31.2	-87.8	-74.5	-89.7
42.00	-30.2	24.8	-29.3	-158.9	-145.5	-164.5
49.00	-31.2	21.9	-31.2	-215.5	-203.1	-220.2
56.00	-30.2	22.8	-31.2	-238.6	-226.2	-244.2

Table B.9. Nebraska sand replacement concrete shrinkage results.

Time (days)	AS1 (με)	AS2 (με)	AS3 (με)	DS1 (με)	DS2 (με)	DS3 (με)
0.00	0.0	0.0	0.0	0.0	0.0	0.0
0.21	7.0	5.2	6.5	3.9	2.7	12.0
0.88	27.9	22.9	28.0	31.6	24.0	36.5
1.03	58.4	50.0	59.3	52.3	47.7	59.1
2.00	58.7	51.9	55.2	76.1	69.4	81.3
5.00	74.1	67.2	70.2	94.4	89.6	100.8
7.00	75.2	68.2	70.8	100.8	95.3	108.4
14.00	91.5	83.0	86.2	120.3	114.1	127.3
21.00	93.1	84.3	87.3	121.4	115.3	128.0
28.00	99.7	90.5	92.5	128.4	122.7	133.6
35.00	104.5	96.3	97.3	60.3	56.4	70.3
42.00	105.5	97.2	100.2	17.1	15.2	28.0
49.00	102.6	96.3	100.2	-20.4	-20.4	-5.6
56.00	102.6	99.2	106.0	-34.8	-34.8	-18.0